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# **Sedimentation Study and Flume Investigation, Mission Creek, Santa Barbara, California; Corte Madera Creek, Marin County, California**

## **Numerical and Hydraulic Model Investigation**

Ronald R. Copeland, Darla C. McVan, and  
Scott E. Stonestreet

April 2000



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# **Sedimentation Study and Flume Investigation, Mission Creek, Santa Barbara, California; Corte Madera Creek, Marin County, California**

## **Numerical and Hydraulic Model Investigation**

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# Preface

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This sedimentation study for Mission Creek in Santa Barbara, CA, and the flume investigation for Corte Madera Creek in Marin County, CA, were conducted jointly at the Hydraulics Laboratory, Vicksburg, MS, of the U.S. Army Engineer Research and Development Center (ERDC), and the U.S. Army Engineer District, Los Angeles (SPL), at the request of SPL and the U.S. Army Engineer District, Sacramento (SPK).

At ERDC, this investigation was conducted during the period July 1990 to September 1991 under the direction of Messrs. Frank A. Herrmann, Jr., Director of the Hydraulics Laboratory; R. A. Sager, Assistant Director of the Hydraulics Laboratory; Marden B. Boyd, Chief of the Waterways Division (WD), Hydraulics Laboratory; Glenn Pickering, Chief of the Hydraulic Structures Division (HS), Hydraulics Laboratory; Michael J. Trawle, Chief of the Math Modeling Branch (MMB), WD; and Bobby Brown, Chief of the Hydraulic Analysis Branch (HAB), HS. The project engineers at ERDC were Dr. Ronald R. Copeland, MMB, and Ms. Darla C. McVan, HAB.

At SPL this investigation was conducted during the period June 1990 to January 1994 under the direction of Messrs. Robert Koplin, Chief of the Engineering Division; Joseph Evelyn, Chief of the Hydrology and Hydraulics Branch; and Brian Tracy, Chief of the Hydraulics Section. The project engineer at SPL was Mr. Scott E. Stonestreet, of the Hydraulics Section. The project engineer at SPK was Mr. Charles Mifkovic.

This report is being published by the Coastal and Hydraulics Laboratory (CHL), Vicksburg, MS. The CHL was formed in October 1996 with the merger of the Coastal Engineering Research Center and Hydraulics Laboratory. Dr. James R. Houston is the Director of the CHL.

During the course of this study, close working contact was maintained among engineers at SPL, SPK, and ERDC. The authors of this report were Dr. Copeland, Ms. McVan, and Mr. Stonestreet.

At the time of publication of this report Dr. Lewis E. Link was Acting Director of ERDC, and COL Robin R. Cababa, EN, was Commander.

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# Conversion Factors, Non-SI to SI Units of Measurement

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Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic feet per second	0.028	cubic meters per second
feet	0.3048	meters
inches	25.4	millimeters
miles (U.S. statute)	1.609347	kilometers
pounds (force) per square foot	47.88026	pascals
pounds (mass)	0.4535924	kilograms
square miles	2.589998	square kilometers
tons per day	907.18	kilograms per day

# 1 Introduction

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## Project Description

The U.S. Army Engineer District, Los Angeles (1986) proposed flood protection for the lower reach of Mission Creek, Santa Barbara, CA (Figure 1). A rectangular concrete channel was designed to convey the 100-year peak flow of 7,900 cfs at supercritical flow. The minimal right-of-way along the channel alignment necessitated use of the supercritical rectangular channel. Two sections of Mission Creek had been improved with concrete-lined, supercritical trapezoidal channels by the California Department of Transportation (Cal Trans) in 1934 and 1964 as indicated in Figure 2. To date, the existing channels have functioned satisfactorily without significant maintenance requirements. The upstream end of the proposed Corps channel would join the existing downstream concrete channel and convey flow 1.1 miles through residential and commercial areas of Santa Barbara to the Pacific Ocean. No facility for trapping sediment was planned at the upstream end of either the existing or proposed concrete channels.

The Mission Creek watershed comprises about 11.5 square miles and is located in a narrow coastal area extending from the Santa Ynez mountains on the north to the Pacific Ocean on the south. The study area is shown in Figure 1. Mission Creek rises about 3,750 ft<sup>1</sup> in elevation and flows about 8 miles to empty into the Pacific Ocean. At approximately the 500-ft elevation, the creek is joined by its main tributary, Rattlesnake Creek. In the headwater areas, stream gradients are as steep as 2,600 ft per mile and average 1,000 ft per mile. In the lower reaches, on the alluvial plain below the foothills, average slopes are about 150 ft per mile. A profile of Mission and Rattlesnake Creeks is shown in Figure 2.

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<sup>1</sup>All elevations (el) cited herein are in feet referred to mean sea level (msl) based on the 1960-78 tidal epoch. These elevations can be adjusted to National Geodetic Vertical Datum (NGVD) using the following equation: NGVD = MSL + 0.06 ft

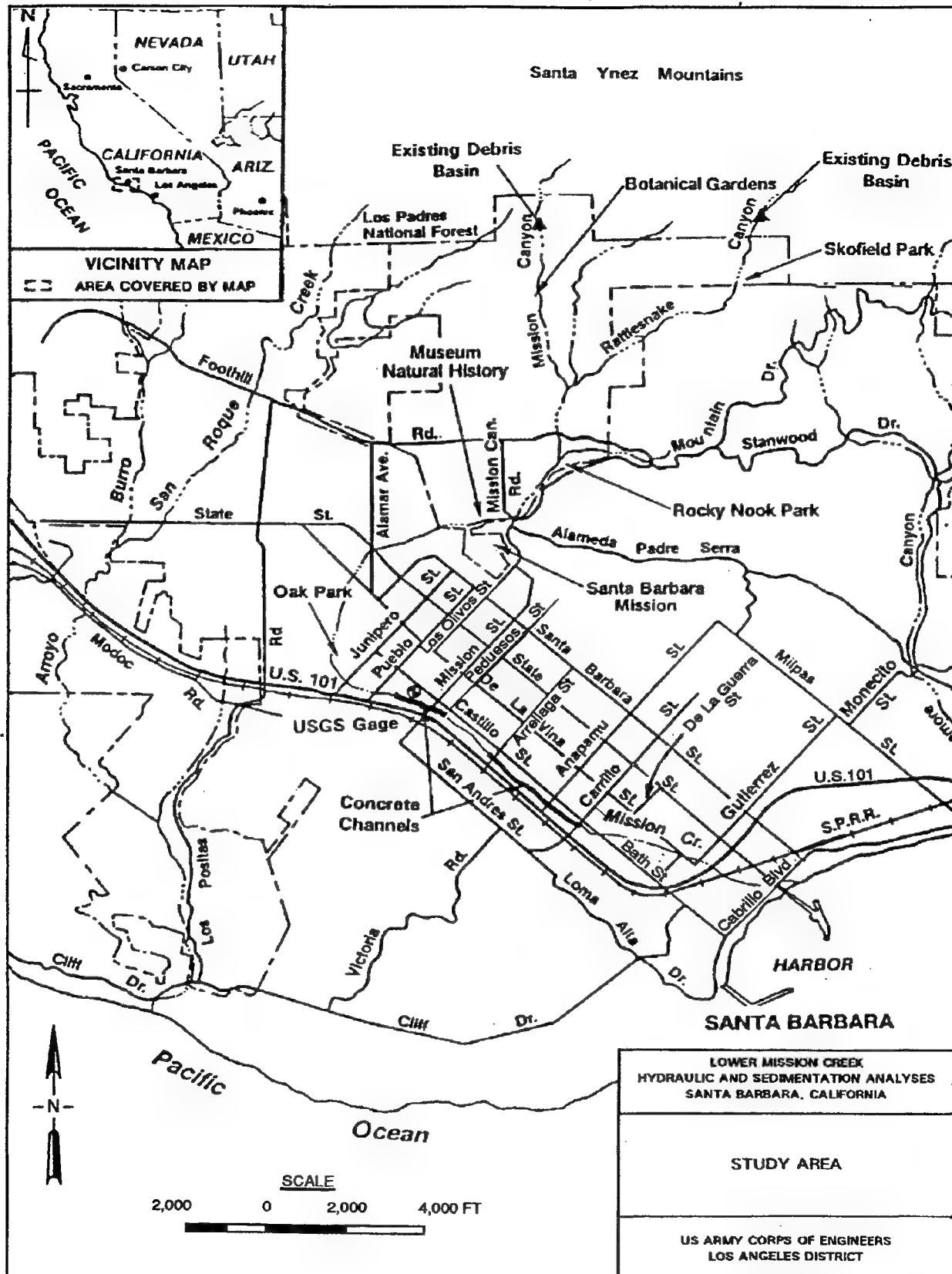


Figure 1. Location and vicinity map

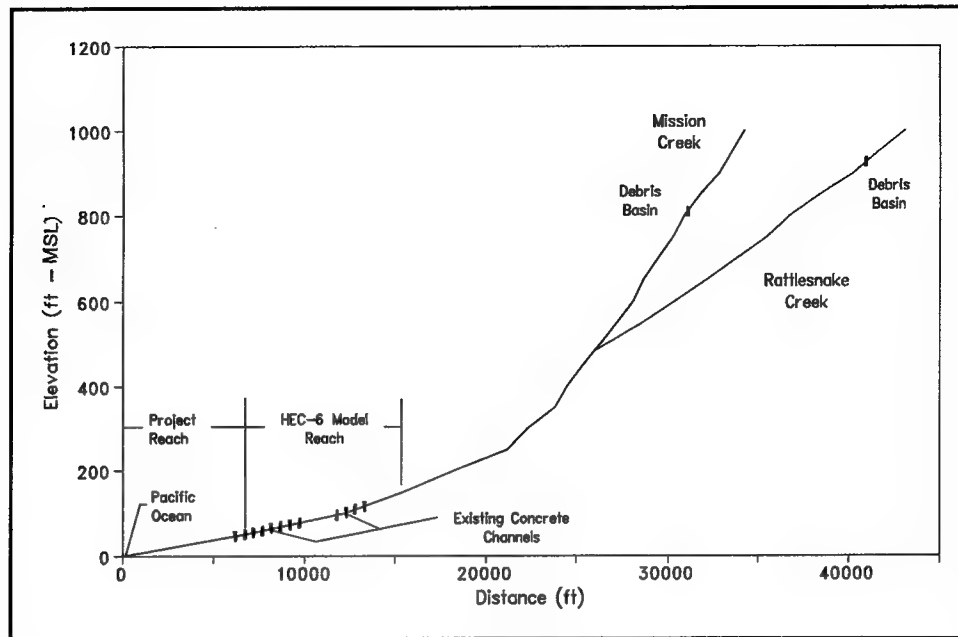


Figure 2. Thalweg profile of Mission and Rattlesnake Creeks

The Mission Creek watershed is capable of supplying large-sized material for sediment transport. Since a debris basin is not included as part of the project, boulders with diameters up to 305 mm may enter the channel. This material, moving as bed load along the channel bottom, could increase the hydraulic roughness to the point at which the flow regime could change from supercritical to subcritical flow. In 1964, the Los Angeles District constructed two small debris basins in the upper reaches of Mission and Rattlesnake Creeks. These debris basins were constructed as an emergency measure to reduce debris and sediment delivery following a fire in the upper watershed. The locations of these basins are shown in Figures 1 and 2. These basins are relatively small and would not provide significant protection to the project reach for the design flood. Additionally, the basins are separated from the project reach by about 4 miles of natural channel with erodible bed and banks.

Since debris control was not a part of the proposed project, it was necessary to evaluate sedimentation effects on hydraulic roughness and deposition. The effect of bed-load transport on the boundary roughness was a concern, especially if the roughness increased to a point at which the flow regime could change from supercritical to subcritical flow. Another concern was the functional reliability at the downstream end of the channel where littoral drift would form a sand plug across the channel exit. The downstream channel invert is at el -6.0, an elevation that the ephemeral Mission Creek would not maintain by flushing. It was uncertain if the channel would be flushed to its design invert on the rising limb of the design flood and thereby be capable of containing the design flood.

## **Purpose and Scope of the Sedimentation Investigations**

Flume and numerical model investigations were conducted to evaluate the effect of bed-load transport on hydraulic roughness and the characteristics of the bed-load transport itself. The flume study was conducted at the Coastal and Hydraulics Laboratory, Vicksburg, MS, U.S. Army Engineer Research and Development Center (ERDC). In conjunction with the flume study, a numerical sedimentation model study was conducted by the Los Angeles District.

The natural bed of Mission Creek upstream from the proposed concrete-lined channel consists primarily of gravels and cobbles that could be transported into the channel at high flow. Bed-material samples containing significant quantities of these coarse sediments were found at the downstream end of the existing concrete channels, confirming that gravels and cobbles pass through the existing channel.

The concrete-lined channel, designed during the feasibility level planning study, was assigned a Manning's roughness coefficient of 0.014. It was assumed that sediment moving through the supercritical flow channel would not have a significant effect on the total hydraulic roughness. This assumption seemed reasonable based on field inspections of existing concrete-lined sections of Mission Creek where no traces of sediment deposits were observed. However, there are documented cases where the hydraulic roughness has increased in concrete channels due to gravel deposition and/or transport (Swanson and Williams 1988; Williams 1990; Copeland and Thomas 1989).

It is anticipated that flow energy in Mission Creek will be sufficient to prevent the establishment of bed forms at expected rates of bed-load transport. However, it is expected that bed load moving along the bed of the channel at a velocity less than the velocity of the water will introduce some drag and will therefore tend to increase the effective hydraulic roughness. To provide a higher degree of confidence in the hydraulic design, it was decided to transfer the assignment of roughness coefficient from a judgment basis to a more analytical basis. This was the purpose of the Mission Creek sedimentation study. Two characteristics of the sediment regime are required for this analysis. First, the inflowing concentration of coarse sediment had to be determined. This was accomplished using analytical techniques, including a one-dimensional numerical sedimentation model. Secondly, the effect of this concentration on hydraulic roughness had to be measured. This was accomplished using flume tests that simulated prototype conditions based on the Froude scale criteria.

## **Hydraulic Roughness in Gravel Bed Streams**

Hydraulic roughness in the concrete channel can be attributed to three components: grain (skin) friction, bed form drag, and bed-load movement itself. A large body of research addresses grain roughness over immobile beds in gravel

bed streams. Bed form roughness has been recognized as a significant contributor to hydraulic roughness in some gravel bed streams. However, a search of the engineering literature provided no definitive guidance for determining hydraulic roughness due to gravel transport over a smooth surface.

### Grain roughness

Research devoted to grain roughness in gravel bed streams typically focuses on the characteristic particle size approach based on the classic work of Nikuradse (1933). This approach requires the determination of an effective roughness from the bed material gradation. When one uses a single characteristic grain size to represent bed roughness, the particle size distribution in the gravel bed is assumed to be constant. Particle shape, orientation, packing, spacing, and sorting are also assumed to be constant or insignificant with respect to grain roughness. Equations for friction factor based on grain roughness for fully rough flow take the form

$$\frac{1}{\sqrt{f}} = c_1 + \frac{1}{k\sqrt{8}} \ln \left( \frac{R}{k_s} \right) \quad (1)$$

or

$$\frac{1}{\sqrt{f}} = c_1 + c_2 \log \left( \frac{R}{k_s} \right) \quad (2)$$

where

$f$  = friction factor

$c_1, c_2$  = constants

$6$  = von Karman's turbulent exchange constant (0.4)

$R$  = hydraulic radius

$k_s$  = effective roughness height

Several examples of these friction factor equations for fully rough flow are presented herein.

Keulegan (1938) using data from rough flat experimental channels (Darcy and Bazin 1865) proposed an equation that represents strictly grain roughness.

$$\frac{1}{\sqrt{f}} = 2.21 + 2.03 \log \left( \frac{R}{k_s} \right) \quad (3)$$



The appropriate value for  $k_s$  in Keulegan's equation has been a subject for debate. Einstein (1950) suggested  $d_{65}$ . Where  $d_{65}$  is the grain size diameter for which 65 percent of the sediment in the bed is finer.

Leopold and Wolman (1957), using data from Brandywine Creek, a gravel bed stream in Pennsylvania (Wolman 1955), proposed the following equation:

$$\frac{1}{\sqrt{f}} = 1.0 + 2 \log \left( \frac{D}{d_{84}} \right) \quad (4)$$

where  $D$  = depth

A similar equation was proposed by Limerinos (1970) using data from 11 gravel and cobble bed rivers in California:

$$\frac{1}{\sqrt{f}} = 1.16 + 2.0 \log \left( \frac{R}{d_{84}} \right) \quad (5)$$

Hey (1979) used a more extensive data set (Limerinos 1970; Barnes 1967; Wolman 1955; and 21 sites in the United Kingdom) to develop an equation that takes into account the cross-section shape:

$$\frac{1}{\sqrt{f}} = c_1 + 2.03 \log \left( \frac{R}{d_{84}} \right) \quad (6)$$

where  $1.02 < c_1 < 1.19$  depending on cross-section shape.

Griffiths (1981), using data from 46 New Zealand gravel bed streams in addition to data from the United States (Barnes 1967; Emmett and Seitz 1974; Judd and Peterson 1969; Wolman 1955) and England (Bathurst 1978), proposed this equation for immobile gravel bed rivers using the median grain size of the bed rather than the  $d_{84}$  size:

$$\frac{1}{\sqrt{f}} = 0.760 + 1.98 \log \left( \frac{R}{d_{50}} \right) \quad (7)$$

The equations of Leopold and Wolman, Limerinos, Hey, and Griffiths are similar. They will produce larger roughness values than the Keulegan equation. This is expected because the constant is determined from field data and the total roughness necessarily includes additional roughness associated with the natural variability of channel cross sections, which exists even in a relatively immobile stable gravel bed stream. When friction factors calculated from measured field data are compared to calculated friction factors using these equations, there is always considerable scatter about the prediction curve. There is no determinable "best" equation. The Limerinos equation is the most frequently cited and has

been shown to be applicable to many gravel bed data sets. Bray (1979) found that the Limerinos equation produced satisfactory results based on data from 67 gravel and cobble bed rivers in Alberta, Canada. Glass (1987) used Jarrett's (1984) data from 21 high-gradient cobble and boulder bed streams in Colorado and Bathurst's (1978) data from 3 cobble and boulder bed streams in England to show that the Limerinos equation produced reasonable results for friction factor even in streams with large-scale roughness.

In many gravel bed rivers, where bed load is insignificant, the total hydraulic roughness can be adequately estimated using a grain roughness equation. Limerinos (1970), Hey (1979), Bray (1979), and Bathurst (1985) all concluded that grain roughness was the primary factor affecting roughness in the gravel bed rivers that they studied.

### Form roughness

The second most important contribution to total hydraulic resistance in gravel bed rivers is bed form roughness. The occurrence of bed forms in gravel bed streams has been documented by many researchers (e.g., Galay 1967; Griffiths 1981, 1989; Kuhnle and Southard 1988; Hey 1988; Kuhnle 1992; Dinehart 1992). In both gravel bed flumes and rivers, it has been shown that an increase in hydraulic roughness typically occurs with increases in sediment transport. Interestingly, this is opposite to what typically occurs in sand bed streams (Einstein and Barbarosa 1952) and flumes (Simons and Richardson 1966). In sand bed streams the decrease in roughness with increase in sediment transport is attributed to a reduction in the magnitude of bed forms. In coarse-bed streams, bed form magnitude tends to increase as sediment transport increases. Shen (1962) first noted this using flume data with a coarse sand bed. He reported that when Einstein's sediment mobility parameter  $Q$  exceeded a value of 5.5, hydraulic roughness increased.  $Q$  is defined by

$$\Psi = \frac{r_s - r}{r} \frac{d_{35}}{R S} \quad (8)$$

where

$D_s$  = density of sediment

$D$  = density of water

$d_{35}$  = grain size for which 35 percent of the sediment in the bed is finer

$S$  = slope

Cunha (1967) also documented an increase in hydraulic roughness when  $Q$  exceeded 5.5 using data from the Mondego River in Portugal ( $d_{50} = 2.5$  mm). Gladki (1979), Prestegard (1983), and Griffiths (1989) also documented an increase in hydraulic roughness with increases in sediment transport in gravel

bed rivers. Griffiths found that bed form roughness exceeded grain resistance when the dimensionless shear stress exceeded the critical Shield's stress by a factor of three in subcritical flow and a factor of five in supercritical flow. Prestegard concluded that bed form (bar) resistance accounted for 50-75 percent of the total resistance in the 12 gravel and cobble bed streams she evaluated.

Griffiths (1981) suggested that hydraulic roughness in mobile-boundary gravel bed rivers is primarily a function of a mobility parameter and recommends the following equation to determine friction factor:

$$\frac{1}{\sqrt{f}} = 2.21 \left( \frac{V}{\sqrt{g d_{50}}} \right)^{0.0340} \quad (9)$$

where

$V$  = velocity

$g$  = acceleration due to gravity

### **Roughness due to sediment transport**

The third primary component of hydraulic resistance in gravel bed rivers is due to the drag created by the particles moving along the bed. Raudkivi (1976) states that this is the least explored of the three components of hydraulic resistance. Vanoni (1946) concluded that suspended sediment load reduces hydraulic roughness by dampening the flow turbulence or by interfering with its production near the bed, thus causing sediment-laden water to flow faster than clear water. His conclusion was based on flume studies over a fixed flat bed roughened with sand grains. He used fine sand as suspended sediment. Vanoni measured as much as a 20 percent reduction in friction factor when the suspended sand concentration was 1,200 ppm. Vanoni and Nomicos (1960) confirmed this result for fixed undulating beds. They measured reductions in the friction factor of between 5 and 28 percent using fine sand at concentrations between 3,600 and 8,100 ppm. They also concluded that the increase in roughness due to the formation of bed forms was much more significant than the decrease due to suspended sediment transport. However, Raudkivi (1976) states that experimental data have produced conflicting conclusions with some showing an increase in turbulence intensity with concentration and others a decrease. According to Raudkivi, flume experiments indicate that a small amount of fine sediment in a flow over a smooth boundary has very little effect on friction factor and might marginally lower it, but when the bed is rough, for example with fixed grains, there is always an increase in friction.

Several researchers have demonstrated that sand moving near the bed increases hydraulic roughness in flumes and rivers. These include Smith and McLean (1977), Grant and Madsen (1982), Dietrich (reported in Wiberg and Rubin 1989), and Wiberg and Rubin (1989). These studies indicated that

Nikuradse's roughness parameter  $z_o$  over a mobile bed can be 1 to 3 orders of magnitude larger than stationary bed roughness. Dietrich developed the following equation to determine the roughness parameter as a function of bed shear stress  $J_b$  and grain size  $d_{50}$ .

$$z_o = 0.033d_{50} + 0.077d_{50} \frac{0.6T_*}{1 + 0.2T_*} \quad (10)$$

$$T_* = \frac{t_b}{t_{cr}}$$

where

$J_{cr}$  = critical shear stress

Wiberg and Smith (1991) presented the following relationship for friction factor as a function of roughness parameter.

$$\frac{1}{\sqrt{f}} = 2.03 \left( \log \frac{R}{d_{84}} + \log \frac{0.48d_{84}}{z_o} \right) \quad (11)$$

Nalluri and Kithsiri (1992) conducted fixed rough bed flume studies to determine critical velocity for maintaining deposit-free channels and pipes. As part of this investigation they determined the increase in friction factor due to bed load transport at the critical condition just before deposition began. The size of sediment in transport varied between 0.5 and 8.74 mm. They found that the hydraulic roughness increase was a function of grain size and concentration. They developed a regression equation with an  $r^2$  ( $r$  = correlation coefficient) of 0.964.

$$f_s = 0.851 f^{0.86} C_v^{0.04} d_{gr}^{0.03} \quad (12)$$

$$d_{gr} = \left[ \frac{g(s-1)}{v^2} \right]^{1/3} d_{50} \quad (13)$$

where

$f_s$  = friction factor with sediment transport

$f$  = friction factor with clear water

$C_v$  = sediment concentration by volume

$s$  = specific gravity of sediment

$\nu$  = kinematic viscosity

## 2 Hydraulic Design

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The proposed project discussed in this report is based on the recommended alternative presented in the Feasibility Report (U.S. Army Engineer District, Los Angeles (USAEDLA) 1986). Approximately 1 mile of concrete, rectangular channel would convey the 100-year design discharge from Carrillo Street to the Pacific Ocean (Figure 1). The channel would have a base width varying from 26 to 38 ft. Final wall heights were not determined during the Preconstruction Engineering and Design (PED) phase of the study, but would generally range from 9.5 to 13.0 ft in height.

The hydraulic design is based on theoretical analyses, using standard Corps of Engineers criterion (Headquarters, U.S. Army Corps of Engineers (HQUSACE) 1991). Water surface profiles for the proposed channel were computed using the Los Angeles District WASURO computer program (USAEDLA 1985). The sedimentation investigations presented herein conform to accepted Corps of Engineers methodologies and approaches (HQUSACE 1989).

### Existing Hydraulic Structures

In the study reach, three sections of the channel have been previously improved. Downstream from Oak Park are the first two improved sections. These are trapezoidal, concrete-lined channels constructed by the California Department of Transportation (Cal Trans) for supercritical flow with an estimated capacity of about 7,000 cfs (USAEDLA 1986). The upstream channel is about 0.3 miles long and extends between Los Olivos and Pedregosa Streets. Downstream is the second channel, which is about 0.8 miles long extending between Arrellaga and Carrillo Streets. The Cal Trans channels have a base width of 26 ft and side slopes of 1V:1H. The slope of the invert is approximately 0.0133 or 70.2 ft/mile. The most downstream channel improvement is a rectangular channel, with a concrete invert and stone sidewalls. This section begins at the Southern Pacific Railroad crossing and extends about 0.1 mile downstream. The estimated capacity of this subcritical channel is about 3,000 cfs.

Additionally, many short reaches of Mission Creek are lined with piled stone, sacked concrete, gabions, and pipe and wire revetment to prevent localized bank erosion and flooding of adjacent residential or other developments.

## Design Discharges

The 100-year flood discharges for lower Mission Creek will vary from 7,500 cfs near Carrillo Street to 7,900 cfs at the mouth of the channel near Cabrillo Boulevard. The design discharges were prorated along the channel based on the location and capacity of existing storm drains and on the location of existing catch basins that drain directly into the channel. The final proration of design discharges is tabulated as follows:

Location	Local Inflow (cfs)	Total Discharge (cfs)
Upstream end of Project		7,500
Canon Perdido Street	50	7,550
De La Guerra Street	50	7,600
Bath Street	50	7,650
De La Vina/Haley Streets	75	7,725
US 101	100	7,825
Mason Street	75	7,900
Pacific Ocean		

Additional discharge information was required upstream of the proposed project reach to set the correct starting depth since flow entering the project will be supercritical. The upstream hydraulic control point (i.e., location of critical depth) would be located at the upstream end of the Cal Trans concrete-lined channel near Arrellaga Street. In this reach, inflow to the channel would include 6,600 cfs from upstream and 900 cfs from local inflows. The local inflows would occur primarily at four storm drains and by sheet overflow originating in the subarea south and west of U.S. Highway 101. The location, size, and assumed inflow from these sources are shown in the following tabulation:

Location	Size & Type <sup>1</sup>	Side	Total Inflow ) Q (cfs)	Total Discharge (cfs)
Inflow at Arrellaga Street				6,600
Sola Street	(42") CP	Left	80	
Victoria Street	(72") RCP	Left	220	6,680
U/S of Carrillo Street	(60") RCP	Right	400	6,900
Carrillo Street	Sheet Overflow	Right	140	7,300
D/S of Carrillo Street	(36") RCP	Left	60	7,440
Inflow to project reach				7,500
<sup>1</sup> CP = corrugated pipe; RCP = reinforced concrete pipe.				

## Proposed Channel

The proposed channel would consist of a collector system for overflow side inflows, a concrete rectangular channel, and an outlet structure located at the downstream end of the channel. The rectangular concrete channel would tie into the existing trapezoidal channel constructed by Cal Trans at the Carrillo Street bridge.

The proposed channel would be 5,970 ft in length and would have a rectangular cross section. Base widths would vary from 26 to 38 ft. In general, the proposed channel alignment follows the existing stream except for a short reach near the U.S. Highway 101 and Southern Pacific Railroad crossings (referred to as the "oxbow" area). The channel invert slopes from Carrillo Street to the beach are relatively steep and range from 0.02000 to 0.00524. These invert slopes were designed to provide a smooth vertical transition in slope from the relatively steep slope of the Cal Trans channel to the less steep slopes located near the beach while providing for stable, supercritical flow throughout the proposed channel.

A finalized design of the channel outlet was not developed during this study. However, the outlet would conceptually consist of an abrupt termination of the rectangular channel just downstream of Cabrillo Boulevard. As shown in Figure 3, the channel would extend about 65 ft beyond the existing bike path so that the hydraulic jump would occur on the beach and not migrate upstream under Cabrillo Boulevard. The end of the channel would include a cutoff wall and a layer of dump stone to preclude undermining of the concrete channel. Additionally, tie-back walls, projecting at a 45 degree angle from the downstream end of

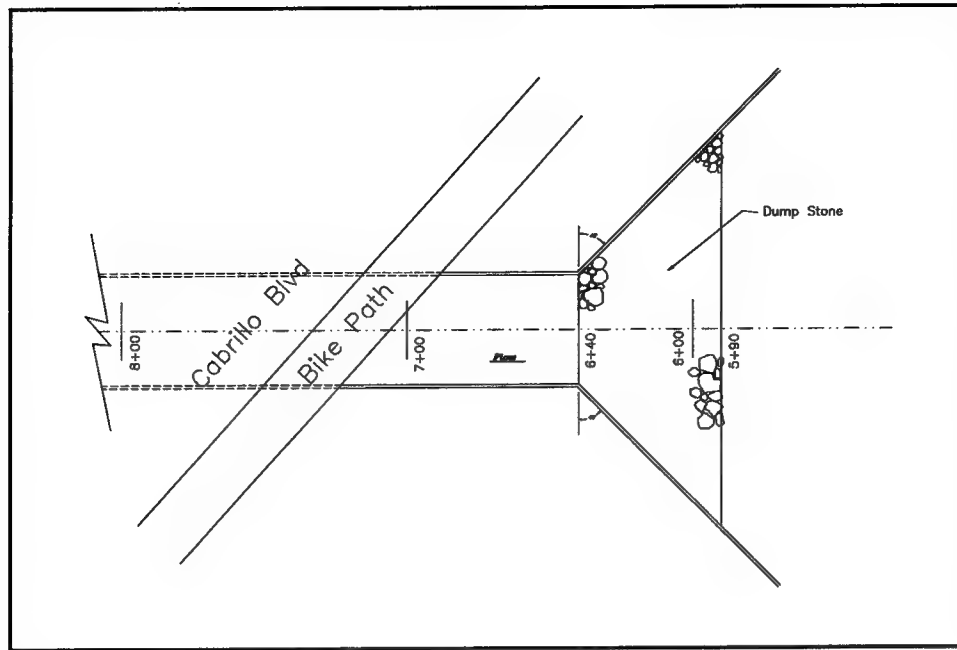


Figure 3. Proposed channel outlet at beach

the channel walls, would be constructed to preclude flows behind the channel walls and to act as retaining walls for backfilling of the surrounding grade.

Given the expected bed-load transport of the channel, it was important to preclude rapid changes in the invert slope, which could act as sediment traps for boulders and cobbles. Proposed channel grades and base widths are tabulated as follows:



Station (5/6/1992 Alignment)	Invert Slope	Invert El	Base Width
67+65 (u/s edge of Carrillo)	0.01330	32.95	30
66+45	Grade Change 0.00900	31.35	30
59+80	Grade Change 0.02000	25.37	Transition
59+00	Grade Change 0.00650	23.77	26
53+40	0.00650	20.13	Transition
53+20	0.00650	20.00	28
39+50	0.00650	11.09	Transition
39+30	0.00650	10.96	30
39+00	Grade Change 0.00524	10.77	30
29+60	0.00524	5.84	Transition
28+80	0.00524	5.42	38
7+00		-6.00	

At the upstream end of the project, the invert elevation and slope were based on the existing Cal Trans channel. At the downstream end of the project, the invert elevation was based on the elevation of the State Street and Cabrillo Boulevard bridge soffits and the depth required to pass 7,900 cfs plus an allowance for freeboard. This requirement resulted in an invert elevation of -6.0 at the beach (sta 7+00).

All of the transitions were designed with recommended rapid-flow transition rates. The transitions converge or diverge on a straight line at a rate not less than 1:20 (horizontal to longitudinal). Two of the transitions have a larger flare ratio to maintain energy requirements for stable supercritical flow in the channel.

## Initial Water Surface Computations

The initial water surface profile was computed by the reach method using the Manning formula to estimate friction losses. The computer program WASURO (USAEDLA 1985) was used to make the calculations. The flow regime in the channel is supercritical beginning with a critical depth control located at the upstream end of the Cal Trans channel (near Arrellaga Street). At the peak flow

rate, a hydraulic jump would occur downstream of the channel outlet if there are no appreciable sediment deposits in the channel. However, results of the sedimentation investigation indicated that significant sediment deposits may be present in the channel near the outlet, which force the hydraulic jump to move upstream into the channel and cause flow to overtop the channel walls. Refer to Chapter 3 for further details.

The initial design water surface profile was computed using a Manning's roughness coefficient ("n-value") of 0.015 for the concrete channel (see "Roughness Coefficients"). Additionally, transition losses were computed using loss coefficients of 0.1 and 0.2 for contraction and expansion, respectively. The design tailwater elevation used for the downstream end of the channel was 2.5, which corresponds to the mean higher high water (mhhw) level of the ocean.

## **Roughness Coefficients**

After the feasibility study was concluded, a concern was expressed regarding the effect of the bed-load transport on the hydraulic roughness of the channel. The specific concern was that gravels and cobbles, transported as bed load, may induce additional drag to the flow, which would increase the effective hydraulic roughness to the point at which supercritical flow could not continue. Since the channel would be designed for supercritical flow, an unexpected subcritical flow regime could overtop the channel, resulting in failure of the project in terms of conveying the 100-year discharge.

# **3 Sedimentation Investigation**

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## **Sedimentation Analyses**

The Mission Creek watershed is capable of supplying large material for sediment transport. Since a debris basin is not included as part of the project, boulders with diameters up to 305 mm could enter the channel. This material moving as bed load along the channel bottom, could increase the hydraulic roughness to the point at which the flow regime may change from supercritical to subcritical flow. Additionally, the impact of the sediment on the performance of the channel near the outlet is unknown and could lead to failure of the project.

### **Purpose**

Flume and numerical model investigations were conducted in support of the Lower Mission Creek PED Study. The flume study was conducted at the Coastal and Hydraulics Laboratory (CHL), now part of the U.S. Army Engineer Research and Development Center. In conjunction with the flume study, a numerical sedimentation model study was conducted by the Los Angeles District and reviewed by CHL.

The purpose of these investigations was twofold: to evaluate the effect of bed-load transport on hydraulic roughness and the characteristics of the bed-load transport itself; and to determine the impact of the sediment load on the performance of the proposed channel near the outlet.

Two detailed numerical sediment model studies were performed to address these concerns. The first sediment analysis modeled the stream system from the Oak Park area to the Cal Trans channel near Carrillo Street and was used to determine the approximate concentration of the bed load (i.e., gravel, cobbles, and boulders) flowing into the proposed concrete channel during the 100-year flood (refer to section "Sediment Analysis for Bed-Load Concentration"). The second sediment analysis modeled the stream system from the Oak Park area to the Pacific Ocean. This model incorporated the previous model as the sediment

source for the proposed channel and was used to analyze the performance of the proposed channel outlet (refer to section "Sediment Analysis of Outlet").

### **Description of the model**

The HEC-6W one-dimensional numerical sedimentation program was used to develop the numerical models for this study. The numerical model allows for transport of individual grain sizes larger than 64 mm along with sand and gravel sizes. The program produces a one-dimensional model that simulates the response of the riverbed profile to sediment inflow, bed material gradation, and hydraulic parameters. The model simulates a series of steady-state discharge events, their effects on the sediment transport capacity at cross sections, and the resulting degradation or aggradation. The program calculates hydraulic parameters using a standard-step backwater method assuming subcritical flow. It assigns critical depth for water surface elevation if the backwater calculations indicate transitions to supercritical flow. However, for supercritical flow, hydraulic parameters for sediment transport are calculated assuming normal depth in the channel. This methodology allows for reasonable sediment transport rates through the supercritical flow channel, but will somewhat overestimate sediment transport capacity in supercritical flow reaches where normal depth has not been reached.

For numerical sedimentation models to simulate the behavior of a stream channel completely, computations would have to account for all of the basic processes of sedimentation: erosion, entrainment, transportation, deposition, and compaction of both the bed and the streambanks for the complete range of particle sizes found in nature. The state of the art has not yet advanced to such a complete simulation. The computer program used in this study, HEC-6W, is a state-of-the-art program for use in mobile bed channels. It is designed to calculate aggradation and degradation of the streambed profile. When applied by experts using good engineering judgment, the HEC-6W program will provide good insight into the behavior of mobile bed channels such as Mission Creek.

Particle sizes from sand to boulders are involved in Mission Creek, which complicates the simulation because particle size controls the fundamental processes in river sediment behavior. The time scale of interest is from a single flood event to the life of the project. The long-term trends can be evaluated from a statistical analysis of the gauge records, but a great deal of variation in water and sediment runoff occurs from one storm event to the next because of the stochastic nature of the hydrologic cycle. The approach for bridging these gaps is to formulate

- a. A procedure that includes the statistical nature of the boundary conditions. The uncertainty in forecasting future hydrology and sediment yield is probably more significant than gaps in modeling the physics of the mobile boundary processes so far as the accuracy of results is concerned.

- b.* A computer program that emulates the physical processes in the project reach sufficiently well to quantify how the sedimentation processes will respond to changes in the boundary conditions and/or to changes in the project geometry or roughness.

Although the sedimentation processes are complex, procedures for describing most of them have been published. The HEC-6W computer program includes those procedures. Where gaps exist between the available procedures, HEC-6W contains logic that bridges those gaps. In summary, it is state-of-the-art technology for calculating the aggradation and degradation in mobile bed channels, and because it has given reliable results at similar projects, it is expected to give reliable answers to the questions being addressed here. All of the HEC-6W computer simulations discussed herein were performed on the CRAY Y-MP supercomputer located at the Information Technology Laboratory at WES.

## Channel geometry

The channel geometry used for the simulations was based on topographic mapping dated 1984 (contour interval 2 ft, scale 1" = 100 ft) compiled for the USAEDLA (1986) Feasibility Study. Additional topographic/geometric data were based on topographic mapping compiled for the PED study (in Intergraph format) dated April 1990 (contour interval 1 ft, scale 1" = 40 ft) and on survey notes developed for the 1990 mapping. Datum for all mapping was mean sea level (msl).

Reach lengths between cross sections are generally greater in a HEC-6W model than in a HEC-2 model. Reach lengths in these models generally range from 158 to about 409 ft in the supply reach and from 198 to 2,000 ft in the concrete-lined reaches of the channel. On the beach, the cross-section spacing was as short as 100 ft and in the ocean averaged about 500 ft. Cross-section locations for the sediment supply reach are shown in Figure 4 and for the outlet reach are shown in Figure 5.

## Hydrology

Discharge hydrographs are simulated in the numerical model by a series of steady-state events also referred to as a histogram. The duration of each event is chosen such that changes in bed elevation due to deposition or scour do not significantly change the hydraulic parameters during that event. At relatively high discharges, durations need to be short. And at low discharges, the time interval may be extended.

For the bed-load concentration analysis, a 60-hr balanced hydrograph, representing the 100-year flood, was modeled as a histogram with a peak discharge of 6,700 cfs in the supply reach. This 100-year histogram is shown in Figure 6 as the solid line and has a constant time interval of 5 min.

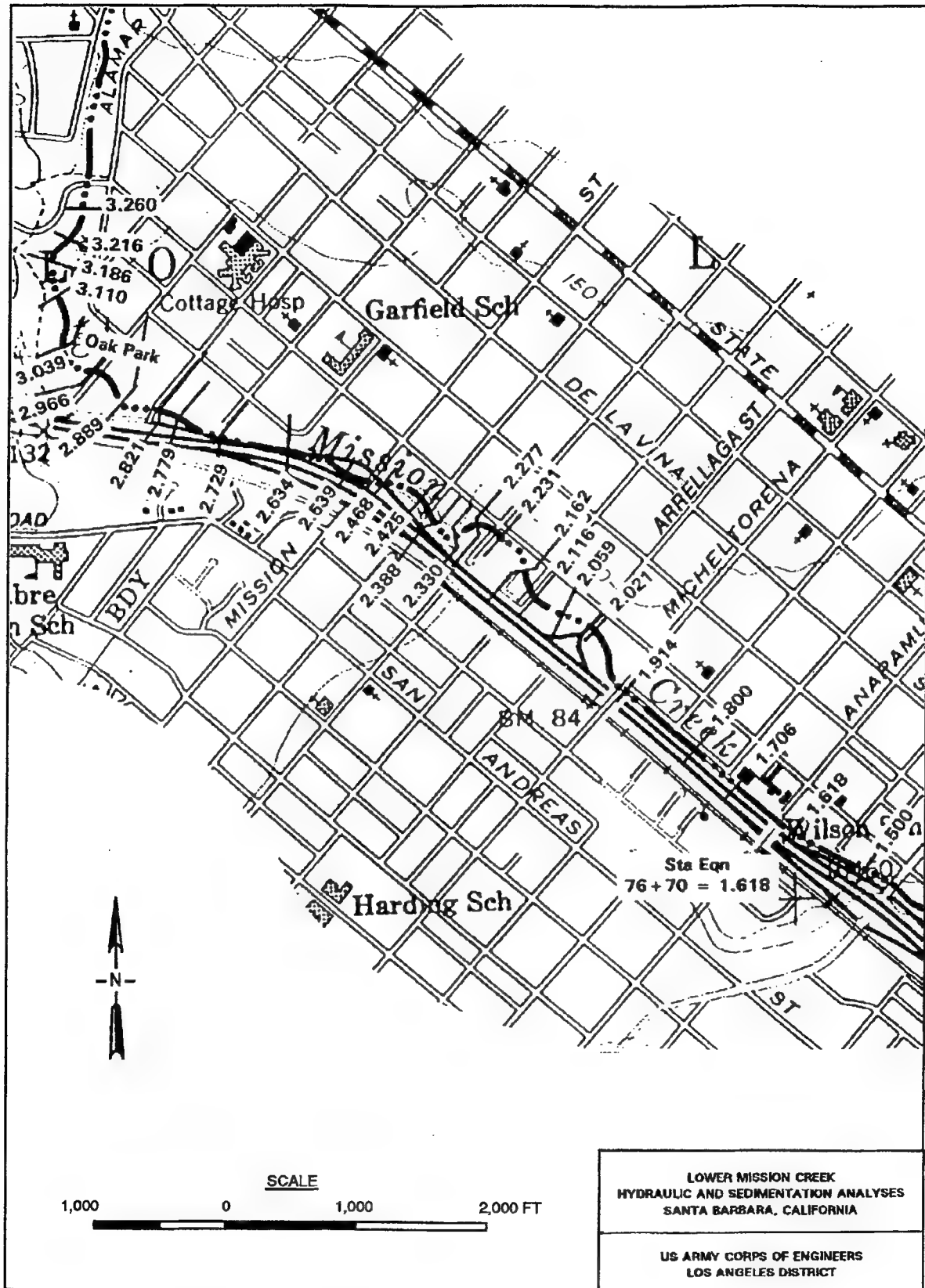


Figure 4. Cross-section locations in sediment supply reach

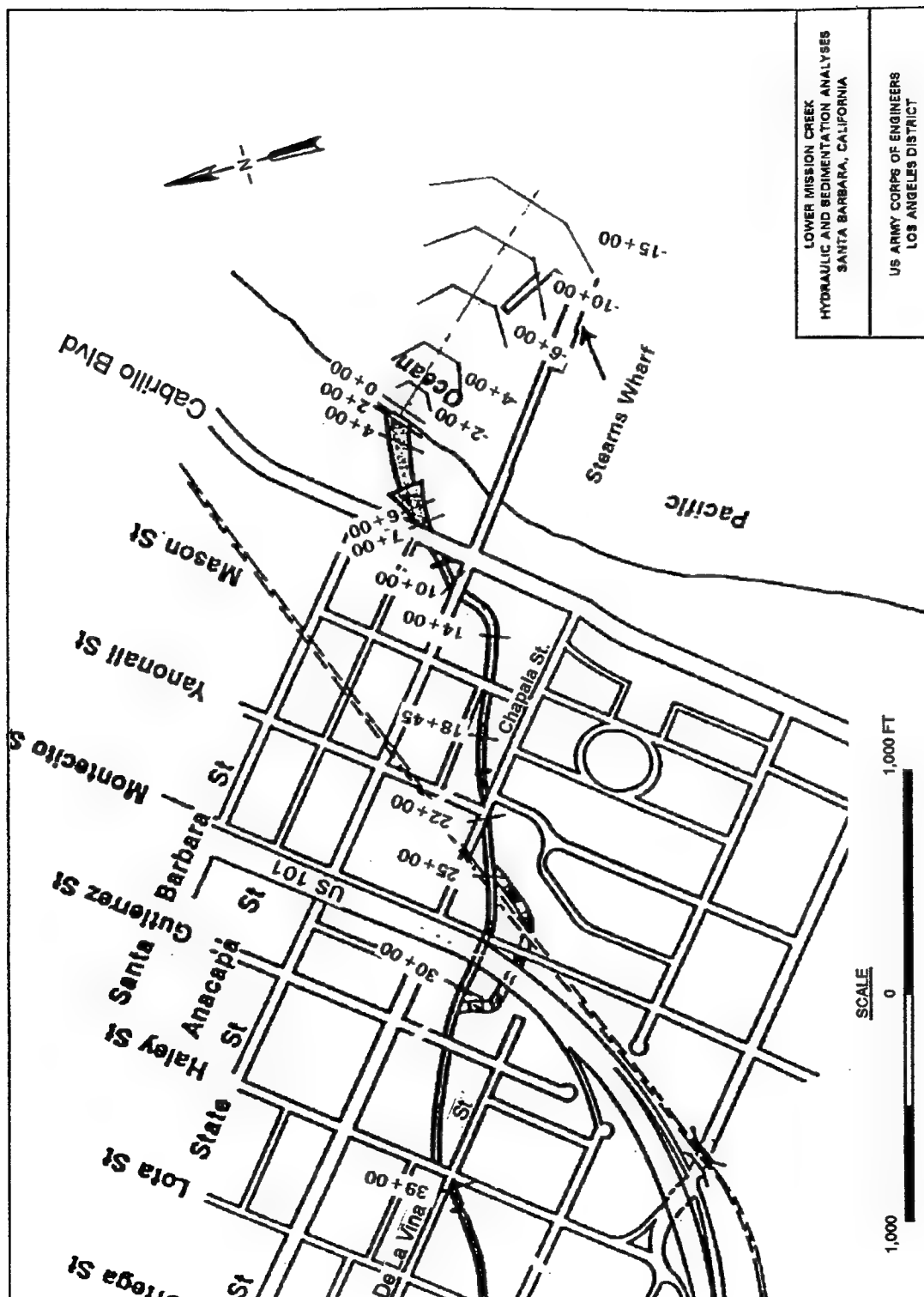


Figure 5. Cross-section locations at channel outlet

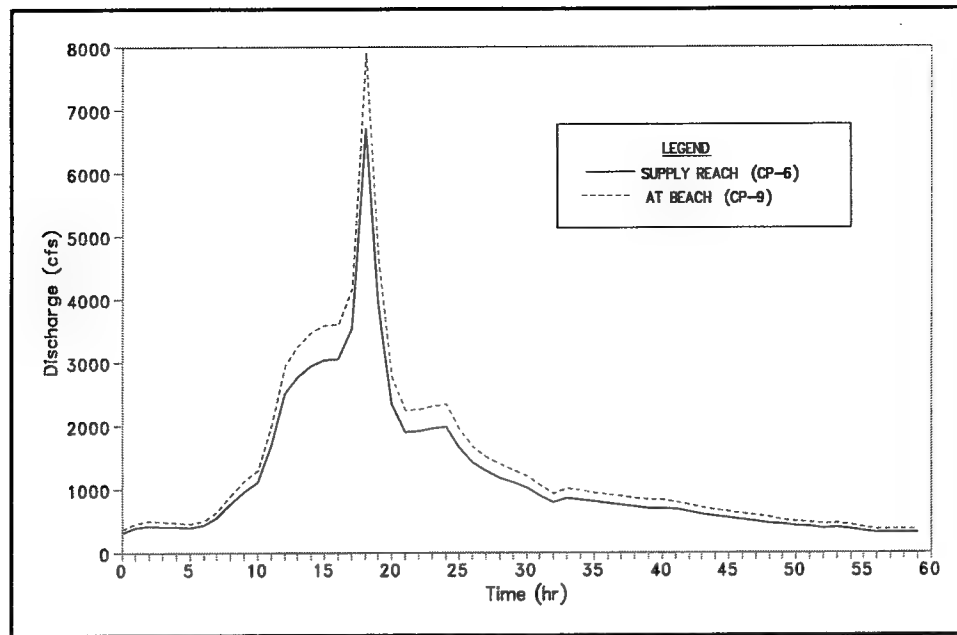


Figure 6. 100-year histograms

For the outlet analysis, additional inflows from the urbanized area downstream of the sediment supply reach were added to the 100-year histogram to develop a total discharge histogram at the channel outlet with a peak discharge of 7,900 cfs, shown in Figure 6 as the dashed line. Additionally, antecedent flows were included in the simulation of the channel outlet to account for the effect of the sediment deposition in the concrete channel prior to the main 100-year flood event. These antecedent flows are discussed in the section "Sediment Analysis of Outlet," "Antecedent flows."

### Sediment yield

During the Feasibility Study, Simons, Li and Associates (1984) conducted a debris yield study for the Los Angeles District. They concluded that, depending on the burn conditions of the watershed, the 1 percent chance exceedance flood could produce between 221 acre-ft (AF) and 1,646 AF at the confluence of Mission and Rattlesnake Creeks. This location is about 2.9 miles upstream of the upstream end of the Cal Trans channel, which the proposed project would tie into. Using sediment transport capacity in the existing channel as a criterion, HEC-6W was used to calculate the bed-material load entering the concrete channel near Arrellaga Street. The calculated 1 percent chance exceedance flood yield ranged between 40 AF and 205 AF. The difference in the computed yields is due to the application of different sediment transport formulae.

The difference between the debris yields of Simons, Li and Associates (1984) and the transport volumes could be accounted for by in-channel storage and lack of available transport capacity as the slopes tend to decrease from upstream to downstream. Either way, it appears a sufficient amount of sediment is available



in the Mission Creek watershed to substantiate concerns regarding the functionality of the proposed channel outlet.

### **Bed material**

Sediment samples were collected along the creek and a gradation analysis was performed to determine the grain size distribution and the maximum particle size. Along the channel, a backhoe was used to collect the sediment samples since the bed material consisted primarily of boulders and cobbles. At the beach, a crane fitted with a clamshell bucket was used to collect samples due to wet conditions at the site. The sediment sample sizes were approximately 1/2 cu yd each. Additional sediment samples were collected by hand at several locations with volumes of 1 to 2 cu ft each. The sediment sample locations are shown in Figure 7.

The sediment gradation of the bed material in the supply reach of Mission Creek is shown in Figure 8. In the supply reach, the maximum grain size of the bed material collected was about 305 mm with an average  $d_{50}$  of about 50 mm. The sediment gradations taken in the channel outlet at the beach are shown in Figure 9. At this location, the maximum size was about 100 mm and the  $d_{50}$  was about 0.35 mm.

Additional sediment data were provided by a debris deposition study performed for the USAEDLA (Simons, Li and Associates 1984). Gradations at several locations along the upper reaches of Mission Creek, well upstream of the project reach, were provided. This gradation analysis, which used the pebble-count method, resulted in a maximum sediment size in the 4- to 8-ft range and a  $d_{50}$  of approximately 254 mm (10 inches). The sediment information from Simons, Li and Associates (1984) was not incorporated into the bed material gradation shown in Figure 8 since these coarse gradations were not representative of the material in the sediment supply reach, specifically downstream of Junipero Street. However, as expected, a review of all sediment gradation data indicates that the maximum and average sediment sizes generally decrease in the downstream direction.

### **Sediment Analysis for Bed-Load Concentration**

The amount of sediment inflowing to the proposed channel was estimated by routing a 100-year balanced hydrograph (histograph) through the natural and concrete-lined Cal Trans channels upstream of the project reach. The sediment routing was performed with HEC-6W. Equilibrium transport was assumed at the upstream boundary of the 1.8-mile study reach (Figure 4).

In this analysis, it is expected that the bed-load material will consist primarily of gravel, cobbles, and boulders. Sand and silt-sized material should be carried as suspended load given the high velocities of the relatively steep stream system.

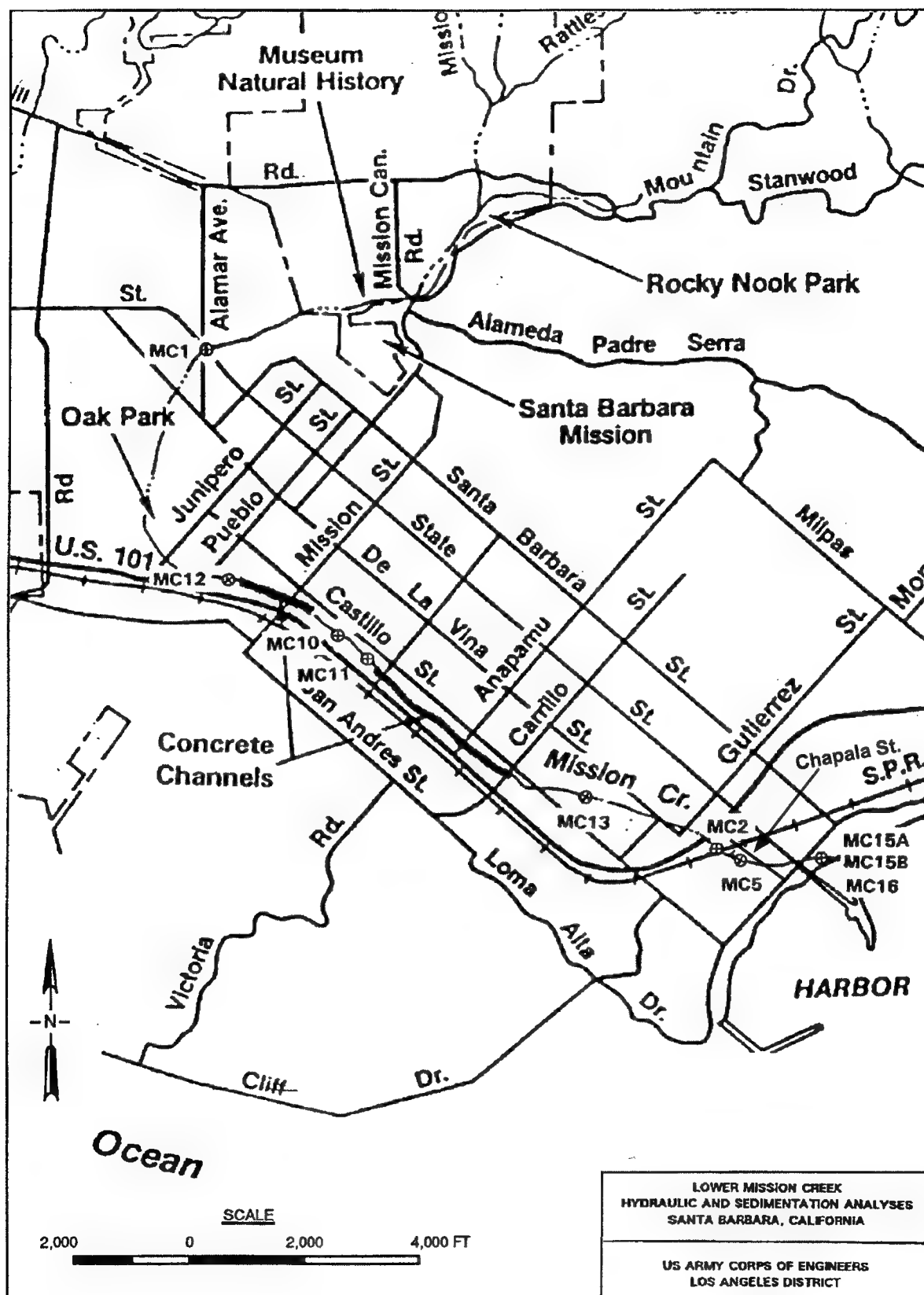


Figure 7. Sediment sample locations

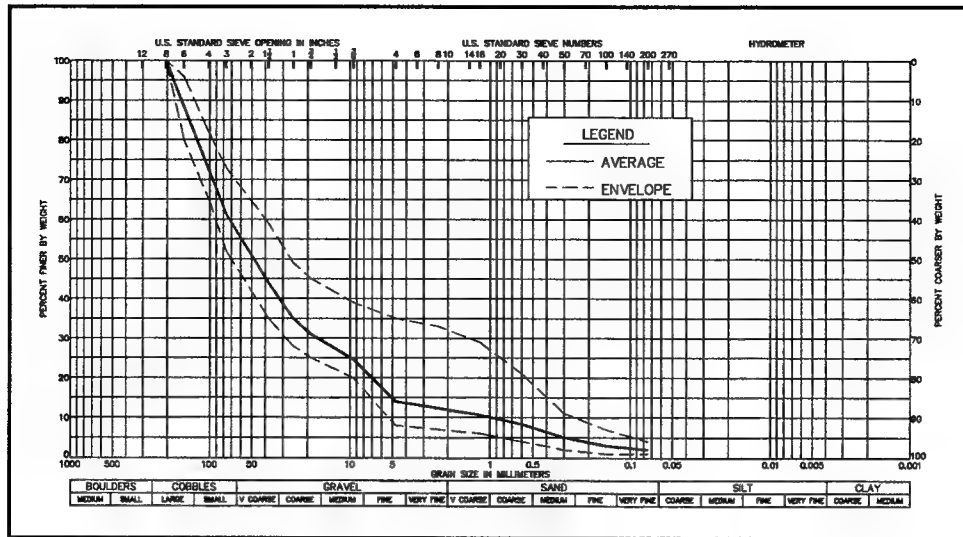


Figure 8. Gradation of bed material in supply reach

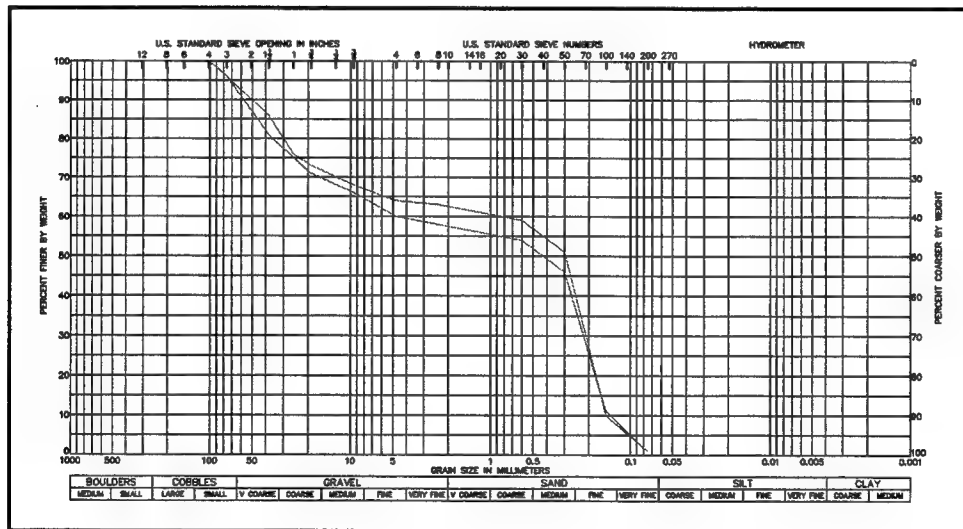


Figure 9. Sediment gradations of the existing channel outlet at the beach

Throughout the sediment supply reach, several constrictive bridges exist in the prototype. In reality, these bridges obstruct flow and may trap significant amounts of sediment and floating debris, reducing the amount of sediment entering the project reach. However, as a conservative measure, the effect of these bridges was not included in the numerical model.

### The model

The numerical model extended from river mile 1.500, located upstream of Carrillo Street in the Cal Trans channel, to river mile 3.260, located in Oak Park. The locations of the cross sections used in this analysis are shown in Figure 4.

Sediment sizes used in this model ranged from very fine sand (VFS) through small boulders (SB). Silt and clay transport was not considered in this analysis.

### **Downstream water surface elevation**

Starting water surface elevations at the downstream end of the numerical model were based on normal depth in the Cal Trans channel upstream of Carrillo Street. This assumption is sufficiently accurate, given the prismatic cross section and nearly constant slope of the existing concrete-lined channel. Additionally, it was assumed that the model was not sensitive to the downstream water surface elevation since critical depth controls exist at the upstream entrances to both of the existing concrete-lined Cal Trans channels.

### **Channel roughness**

Hydraulic roughness is influenced by grain size or bottom roughness, bank or sidewall roughness, bed forms, water depth, changes in channel shape, and changes in flow direction or distribution due to channel bends and confluences. In the one-dimensional numerical model these effects are accounted for by the Manning's roughness coefficient. Acceleration and deceleration of flow are accounted for with expansion and contraction coefficients.

In the sediment supply reach, a Manning's  $n$  value of 0.050 was used based on the conditions existing in the field. This value was based on engineering judgment and on application of Cowan's method (Chow 1959) for estimating Manning's  $n$  values. Through the concrete-lined reaches of the channel, an  $n$  value of 0.014 was used.

### **Transport function**

A combination of the Toffaleti and Meyer-Peter and Müller transport function was used for this study. This combination of functions is desirable since the Toffaleti function is generally applicable for sand bed streams and the Meyer-Peter and Müller function is applicable for gravel bed streams.

Additional transport functions were used in the numerical model and tested for sensitivity, titled "Sensitivity analysis." These functions included the Meyer-Peter and Müller function, a combination of the Toffaleti and Schoklitsch functions, and Yang's unit stream power function.

### **Sediment inflow**

Measurements of suspended or bed-load sediment do not exist for Mission Creek. Therefore, sand and gravel inflow to the numerical model was calculated assuming equilibrium sediment inflow, using average hydraulic parameters in the supply reach and the average bed material gradation shown in Figure 8. By using the average bed gradation, the computations ignore the effect of armoring and the inflow curves tend to be high, especially with a multiple grain size transport function. The equilibrium sediment transport capacities were computed with the

sediment transport module of the Hydraulic Design Package for Flood Control Channels (SAM) developed at CHL (Copeland et al. 1997).

Sediment inflow curves for the total sediment load are shown in Figure 10 for each transport function. These curves are based on equilibrium transport and include transport of sand, gravel, cobbles, and boulders.

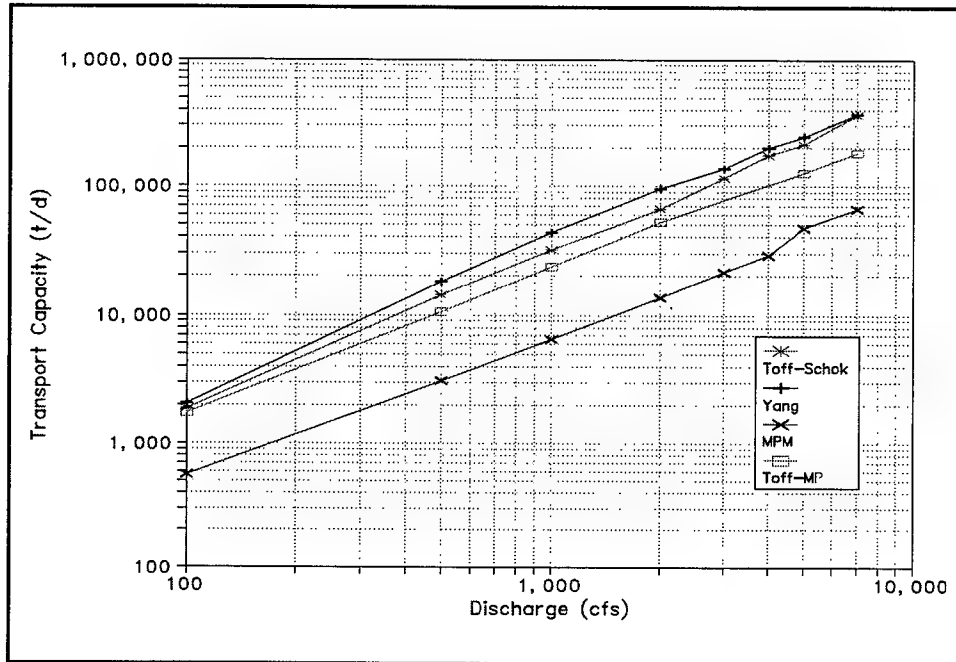


Figure 10. Sediment inflow rating curves for the total sediment load

The gravel and cobble portions of the sediment inflow loads are shown in Figure 11. As illustrated in Figure 11, the Meyer-Peter and Müller function tends to move the largest amount of coarse material while the Yang function moves the least amount of coarse material. In contrast, Figure 10 illustrates that the Yang function moves the largest total load, composed primarily of sand and fine material.

These figures illustrate the rationale for selection of the appropriate sediment transport function(s). Given the emphasis of this analysis was to determine the concentration of the coarse bed load, the Meyer-Peter and Müller function moves the largest amount of coarse material and thus should produce a conservative estimate of the inflowing bed-load concentration. By coupling this function with the Toffaleti function, a function that tends to move a large amount of sand, the effect of a sizable wash load is also taken into account.

### Model adjustment and circumstantiation

Adjustment and circumstantiation of the model was not possible due to a lack of prototype data. This situation is typical of ephemeral streams located in the Southwest. However, to reduce the amount of uncertainty in this or subsequent

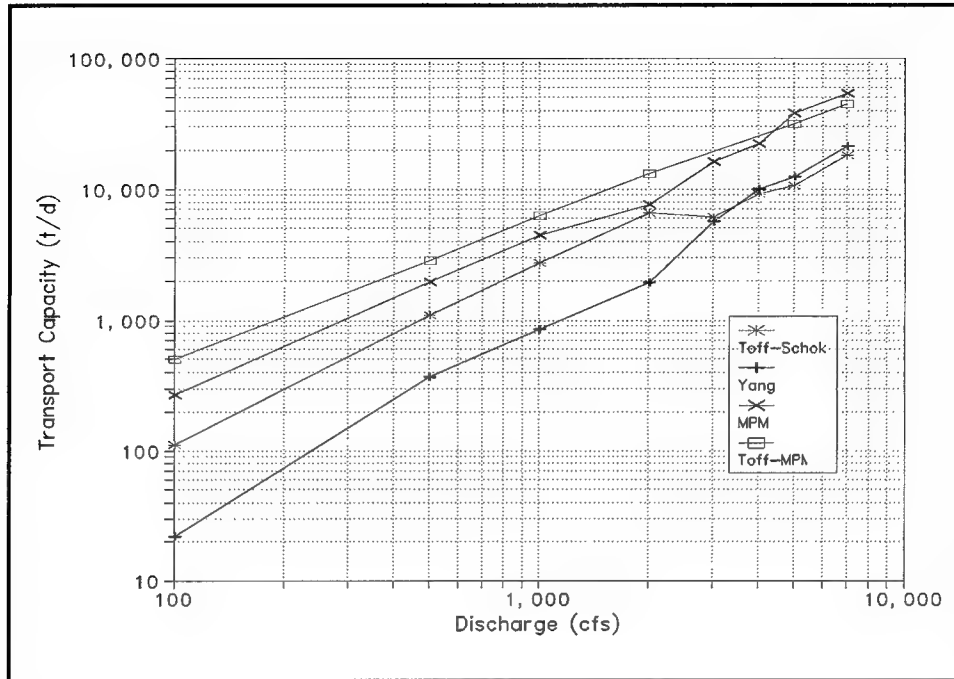


Figure 11. Sediment inflow rating curves for the gravel and cobble fractions of the total load (i.e., bed load)

analyses, a sediment sampling and monitoring program should be developed and implemented for Mission Creek. This program would conceivably consist of collecting suspended and bed material samples from Mission Creek during flood events. Additionally, significant changes in the channel geometry following flood events should be documented in terms of detailed topography. Results from the sampling program could be compared to computed results and used to adjust the numerical model and decrease the uncertainty in the rates and volumes of sediment transport.

## Results

Results for the sediment routing for the peak discharge are shown in Figure 12 and in the following tabulation. These indicate that the concentrations flowing into the concrete channel may vary between 1,450 and 17,900 ppm for the total load, in which the bed-load (gravel, cobbles, and boulders) concentrations vary from 61 to 1,110 ppm. These results are for an inflow sediment load ratio of 1.0. Additionally, it appears that the combination of the Toffaleti and Meyer-Peter and Müller transport functions gives the highest or most conservative bed-load concentration.

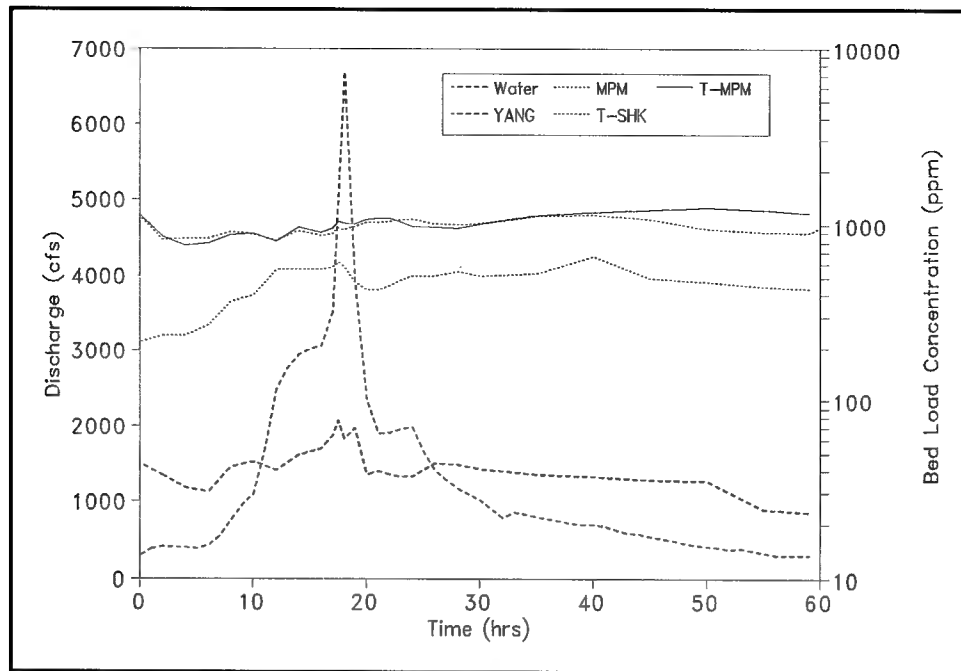


Figure 12. Comparison of discharge during design flood and bed-load concentration for different transport formulae

### Sensitivity analysis

Because of the lack of prototype sediment inflow data, it was especially important to determine the sensitivity of the model to sediment inflow. Simulations of the design flood were conducted using three additional sediment transport equations. As expected, different transport equations produced different inflowing bed-load concentrations. Since suspended sediment data were not available for comparison with calculated transport rates, numerical model results were interpreted considering the sensitivity of the model to the transport function. Additional sensitivity tests conducted included varying the channel roughness coefficient and evaluating possible effects from the imposed initial conditions by accounting for antecedent flows and initial bed gradation. Sensitivity of the model to the downstream water surface elevation was not considered in this analysis since critical depth controls exist at the upstream entrances to both existing concrete-lined Cal Trans channels.

**Sediment inflow.** The sensitivity analysis consisted of testing various sediment transport functions and then varying the inflowing sediment load for each function. The additional transport functions included the Meyer-Peter and Müller function, a combination of the Toffaleti and Schoklitsch functions, and Yang's unit stream power function. The results from these tests are shown in the following tabulation and Figure 12. Additional sensitivity to the sediment inflow was obtained by varying the sediment inflow load. Specifically, the inflowing sediment load was halved and doubled at the upstream end of the numerical model during the simulations using a load ratio option.

Transport Function	Total Load tons/day	Total Concentration ppm	Bed Load tons/day	Bed-Load Concentration ppm
Yang	323,700	17,900	1,100	61
Toff-Schoklitsch	248,800	13,800	10,500	580
Toff-MPM	143,900	7,960	20,000	1,110
MPM	26,300	1,450	17,200	950

As expected, variation in the inflowing sediment load ratio produces slightly different results at the downstream boundary. For example, using the Toffaleti and Meyer-Peter and Müller combination, an inflow load ratio of 0.5 gives a downstream boundary bed-load concentration of about 1,000 ppm, whereas an inflow load ratio of 2 produces a bed-load concentration of about 1,300 ppm (Figure 13). Results from all of the model runs are shown in the following tabulation.

Transport Function	Load Ratio	Total Load		Bed Load	
		tons/day	ppm	tons/day	ppm
T-MPM	0.5	90,170	4,984	21,480	1,187
	1.0	143,900	7,955	20,040	1,108
	2.0	265,900	14,700	19,080	1,054
MPM	0.5	20,690	1,144	15,690	867
	1.0	26,270	1,452	17,170	949
	2.0	24,620	1,361	13,300	735
T-Schk	0.5	165,100	9,126	10,640	588
	1.0	248,800	13,760	10,520	581
	2.0	421,000	23,290	6,530	361
Yang	0.5	164,000	9,064	1,208	67
	1.0	323,700	17,890	1,102	61
	2.0	638,200	35,280	819	45



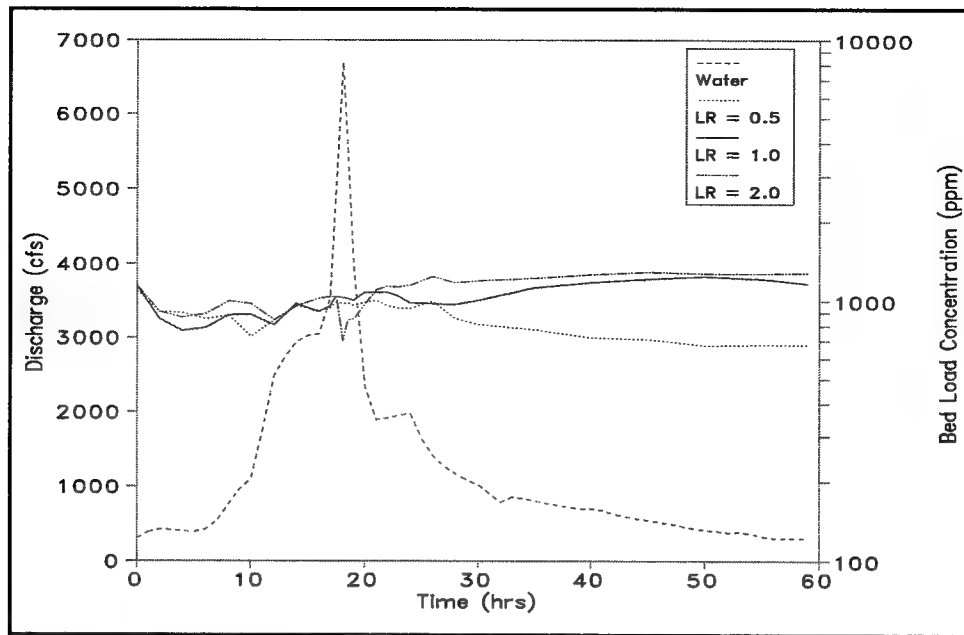


Figure 13. Sensitivity analysis for load ratio for Toffaleti and Meyer-Peter and Müller function

**Antecedent flow.** The impact of antecedent flows on the inflowing sediment concentration was evaluated by simulating two back-to-back 100-year flood events using the Meyer-Peter and Müller transport function. By evaluating this flood combination the sensitivity to antecedent flood events could be evaluated. Additionally, the first 100-year event would provide an initial condition for the second 100-year flood, possibly eliminating numerical instability in the latter case.

Results of this sensitivity test are illustrated in Figure 14. In general, the bed-load concentrations tend to increase slightly during the second 100-year flood but are similar in magnitude to the bed-load concentrations computed during the first 100-year flood. Thus, it was determined that the bed-load concentration was not significantly sensitive to antecedent floods.

## Sediment Analysis of Outlet

The impact of the sediment in the proposed channel near the channel outlet was investigated since the proposed channel design did not include a debris basin. Specifically, an analysis was conducted to determine if the channel outlet would clear the sand plug located at the beach. It was expected that the functionality of the outlet may be hampered by the sediment load conveyed in the channel coupled with the relatively low elevation (i.e., -6.0 ft msl) of the channel invert at the beach.

Similar to the analysis conducted for the bed-load concentration, this analysis was performed by routing a 100-year balanced hydrograph (histograph) through

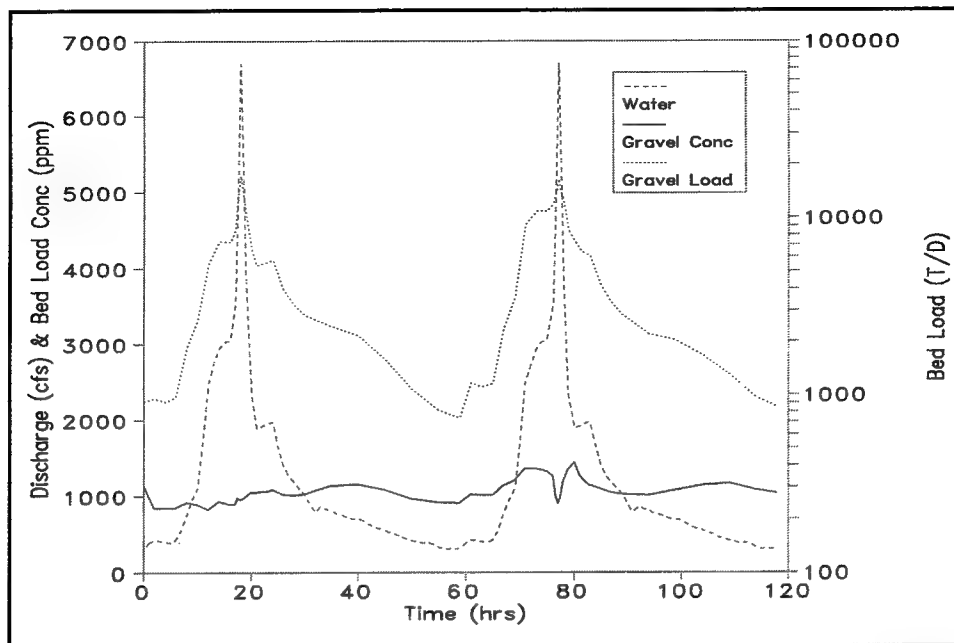


Figure 14. Sensitivity analysis for antecedent flows for Meyer-Peter and Müller function

the proposed concrete-lined channel, the existing concrete-lined Cal Trans channels, and the natural channel upstream of the project reach. The sediment routing was performed with HEC-6W.

### The model

The numerical model extended from river station -15+00, located approximately 1,600 ft offshore (i.e., 2,200 ft downstream of the bikepath at Cabrillo Boulevard), to river mile 3.260, located in Oak Park. Upstream of and including river mile 1.618, the cross sections used for the bed-load concentration analysis are the same as those used for this analysis. A river stationing equation exists near Carrillo Street and was developed to link the river mile stationing upstream (i.e., for the HEC-6W model) with the proposed project stationing. At this location, project station 76+70 equals river mile 1.618. The locations of the cross sections between river station -15+00 and river mile 39+00 are shown in Figure 5.

Cross sections in the ocean were developed using bathymetry from the Santa Barbara 7.5NU. S. Geological Survey (USGS) quadrangle map dated 1952 (photorevised 1967). Some adjustment to the contour lines near the water line of the ocean was required because the width of the beach has grown considerably since the quadrangle map was developed. The detailed topographic mapping dated 1990 was used to adjust the location of the contour lines.

### **Downstream water surface elevation**

The starting water surface elevation at the downstream end of the numerical model was based on the mean sea level for the antecedent flows and on the mean higher high water (mhhw) level of the Pacific Ocean for the simulation of the 100-year flood. In Santa Barbara the mhhw is about 2.5 ft above mean sea level.

The mhhw elevation was assumed to represent a reasonably conservative tailwater level that may include the effect of an increase in water surface due to storm surge activity. The tailwater elevation was maintained at the constant elevations of 0.0 and 2.5 during the entire antecedent and 100-year flood histographs, respectively. Variation of the tide was not included in the model.

### **Channel roughness**

As previously discussed, a Manning's  $n$  value of 0.050 was used in the natural sections of the sediment supply reach. Through the concrete-lined reaches of the channel, an  $n$  value of 0.014 was used with the exception of the lower 1,200 ft of channel. On the beach and in the ocean, a Manning's  $n$  value of 0.025 was used.

Special attention was given to the appropriate roughness coefficient of the lower 1,200 ft of channel near the outlet. This was required since the roughness value may vary greatly depending on the presence of sediment deposits. For example, if the channel is relatively clear of sediment deposits, an  $n$  value of about 0.014 would be appropriate. However, if sediment deposits exist, completely covering the concrete invert, an  $n$  value of 0.025 may be appropriate to account for the increase in roughness due to grain and bed form roughness.

At the time of this study, the version of the HEC-6W model used in this analysis did not have a procedure to simulate the change in hydraulic roughness for a fixed bed (i.e., concrete invert) due to sediment deposition or removal by scour. Thus, the approach used in this analysis assumed an "average"  $n$  value of 0.022 for the channel section near the outlet. This value is based on an assumption that the invert would have some sediment deposits (i.e., possibly due to antecedent events) and would represent a reasonable composite  $n$  value for the bed and channel walls. This  $n$  value was held constant during the entire flood simulation. Sensitivity analyses were conducted to test the impact of the channel  $n$  value on the final results and are discussed in the section "Sensitivity analysis."

It should be noted that the channel geometry used in the model at the beginning of the flood simulation (i.e., prior to the antecedent flows) was based on an assumption that the channel would be clear of sediment deposits due to annual operation and maintenance activities.

## Transport function

Similar to the bed-load concentration analysis, a combination of the Toffaleti and Meyer-Peter and Müller transport function was used for this study. Additional transport functions were used in the numerical model and tested for sensitivity, as discussed in the section "Sensitivity analysis."

## Sediment gradation

Sediment sizes used in this model ranged from VFS through SB. Silt and clay transport was not considered in this analysis. Various bed material gradations were used in the numerical model to account for the relative coarsening of the bed material from the beach to the sediment supply reach near Oak Park. The sediment gradation curves are shown in Figure 15.

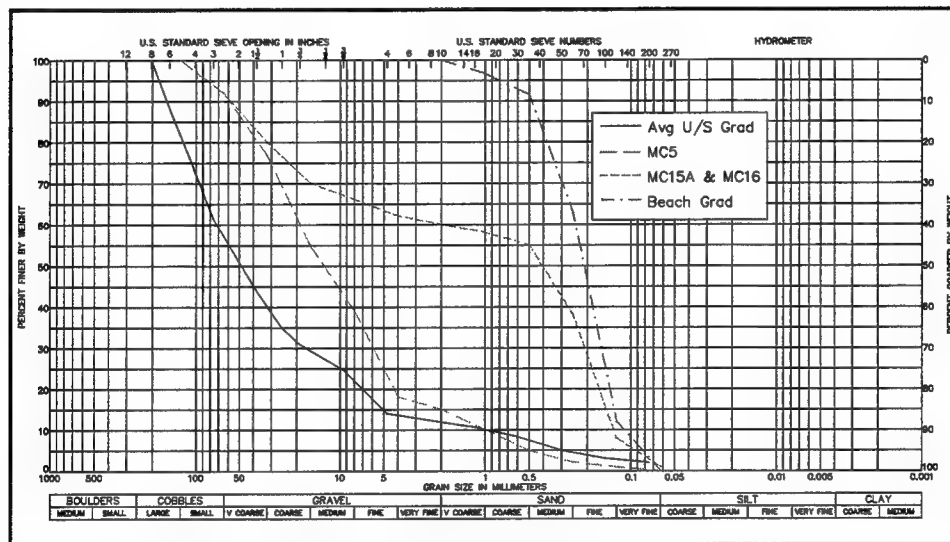


Figure 15. Bed material gradations for outlet analysis

At the beach, the sediment gradation was based on a normalized average of the sand fractions from samples MC15A and MC16. That is, the average sediment gradation from samples MC15A and MC16 was truncated to eliminate the presence of gravels and cobbles and then normalized to 100 percent and used as the representative sediment gradation for the beach. This gradation is shown in Figure 15. This unique approach was required since the only samples of beach material were the samples taken of bed material from the existing channel outlet. These samples included gravel and cobbles, which are not found in the beach sand (i.e., in the sand plug), and were thus removed to produce a gradation more typical of the beach material.

In the proposed channel outlet, the sediment gradation was based on an average of the bed material gradations from samples MC15A and MC16, which included the presence of the gravel and cobble size fractions.

In the proposed channel, between station 7+00 and 66+45, the sediment gradation was based on sample MC5. This sample is from a sediment deposit in the existing concrete/sandstone channel located immediately upstream of Chapala Street. It was expected that this gradation would be representative of deposits that may occur in the lower 1,200 ft of the proposed channel.

Upstream of and including river mile 1.618, the average bed material gradation shown in Figure 8 was used in the numerical model. As discussed in the section "Sediment Analysis for Bed-Load Concentration," this gradation was considered representative of the average bed material gradation in the sediment supply reach.

### **Sediment inflow**

As discussed previously, measurements of suspended or bed-load sediment do not exist for Mission Creek. Therefore, sand and gravel inflow to the numerical model was calculated assuming equilibrium sediment inflow using average hydraulic parameters in the supply reach and the average bed material gradation shown in Figure 8. The equilibrium sediment transport capacities were computed with the sediment transport module of the Hydraulic Design Package for Flood Control Channels (SAM) developed at CHL (Copeland et al. 1997). The total sediment load inflow curve for the Toffaleti and Meyer-Peter and Müller transport function is shown in Figure 10.

### **Antecedent flows**

Antecedent flows were added to the 100-year flood histogram to account for sediment deposition in the concrete channel prior to the main 100-year flood event. The antecedent flows were based on the USGS gauge record for the period of 1 October 1977 through the recession of the 16 January 1978 flood. The hydrograph for the 9 February 1978 flood (i.e., the maximum peak flow rate for the 1978 water year) was not included in the antecedent flows. This sequence of flows was selected since it was observed to be representative of the worst-case antecedent flows for the period of record from the USGS gauge (No. 11119750). A specific antecedent frequency was not assigned to the 1978 flows.

The numerical simulation was conducted with the assumption that any deposition in the concrete channel due to antecedent flows would not be removed prior to the occurrence of the 100-year flood. This assumption was based on operation and maintenance experience in the Los Angeles District for other flood-control channels similar to lower Mission Creek. During prototype operations it was expected that sufficient time may not exist to clear the channel of sediment deposits between major flood events.

### **Model adjustment and circumstantiation**

Similar to the analysis performed for the bed-load concentration, adjustment and circumstantiation of the model were not realistically possible due to a lack of prototype data. However, a cursory study to verify the model was conducted.

This study was based on simulating the sedimentation processes of the existing lower Mission Creek for the period of record from about April 1990 to May 1993. This period was selected since detailed topographic data were available from the 1990 PED mapping (i.e., representing the initial condition geometry) as well as field observations of the channel bed in May 1993 (i.e., representing the ending condition geometry). Additionally, hydrologic data were available from stream gauge records from the USGS gauge located on Mission Creek.

The model simulated all discharges greater than 10 cfs during the period. At the end of the simulation, the computed bed elevations at several bridges were compared with observed elevations (i.e., based on bridge clearances measured in the field). This comparison indicated a general agreement between the simulated results and the prototype. Thus, based on the 4-year period it appeared the model was correctly reproducing the sedimentation processes of lower Mission Creek.

However, due to the short period studied, it should be noted that the results from this cursory study are probably more qualitative than quantitative. That is, the maximum flow rate during the simulated period was 1,300 cfs (from the flood of 15 February 1992) and the maximum flow rate for the 100-year flood was 7,900 cfs. Thus, the model was not verified for large flood events.

To reduce the amount of uncertainty in this or subsequent analyses, a sediment sampling and monitoring program similar to this one should be developed and implemented for Mission Creek.

## **Results**

Results for the sediment routing of the 100-year flood, shown in Figure 16, indicate that sediment will deposit in the channel outlet to a depth of about 5 ft by the peak of the 100-year flood. The sediment deposit will reduce the capacity of the channel outlet to the point at which flood flows will not be contained.

The basic problem is that the channel is required to exit onto the beach at the relatively low elevation of -6.0. This low elevation is required for flows to pass safely under the most downstream bridges (i.e., State Street and Cabrillo Boulevard). Given this low elevation, low flows will tend to flow in the subcritical regime for the downstream 1,200 ft due to the tailwater of the Pacific Ocean. Only the highest flows (e.g., >4,000 cfs) will have sufficient energy to force the hydraulic jump to occur downstream of the channel on the beach.

Thus, sediment is expected to deposit in the concrete channel during low flows. These low flows make up about 99 percent of the total flood hydrograph (i.e., antecedent and 100-year flows). Only during flows of about 4,000 cfs and greater does supercritical flow occur throughout the channel, which would tend to scour and remove sediment deposits. However, these high flows are of insufficient duration to completely clear the channel before the peak flow arrives.

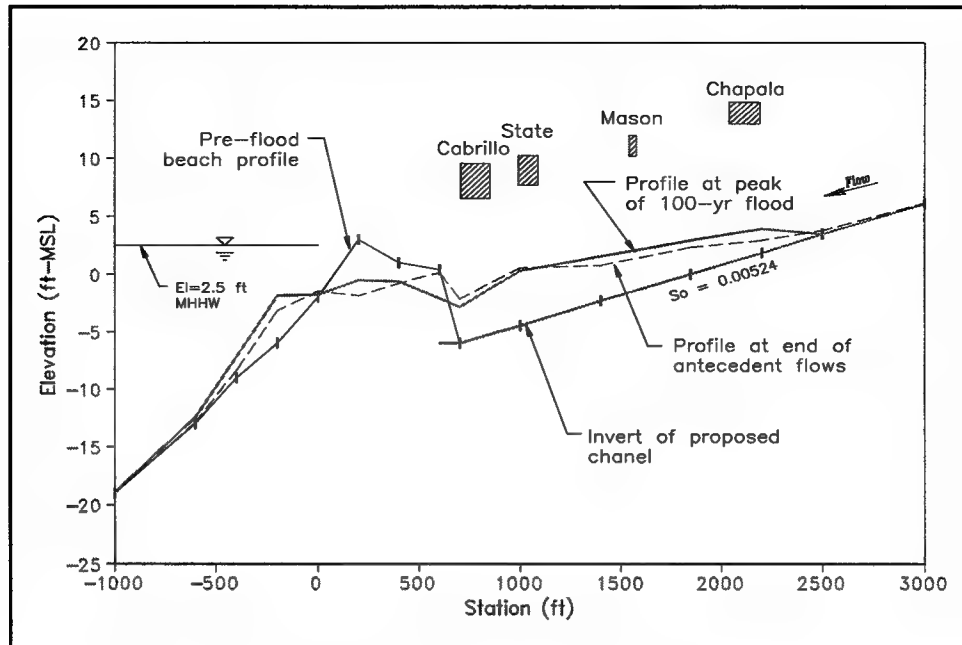


Figure 16. Results of sediment routing at channel outlet

### Sensitivity analysis

Because of the lack of prototype sediment inflow data, it was especially important to determine the sensitivity of the model to sediment inflow. Simulations of the design flood were conducted using four additional sediment transport functions. Since suspended sediment data were not available for comparison with calculated transport rates, numerical model results were interpreted considering the sensitivity of the model to the transport function. Additional sensitivity tests conducted included varying the channel roughness coefficient near the channel outlet and evaluating possible effects from the downstream water surface elevation.

**Sediment inflow-transport function.** The sensitivity analysis consisted of testing various sediment transport functions. The additional transport functions included a combination of the Toffaleti and Schoklitsch functions, Yang's unit stream power function, Madden's 1985 modification to Laursen's (1958) function, Madden (1993), and Copeland's modification to Laursen's (1958) function (Copeland and Thomas 1989). Madden's 1985 modification of Laursen's function adapted Laursen's function for higher Froude numbers and included Toffaleti's river data and Guy, Simons, and Richardson's 1966 flume data. Copeland's modification to Laursen's (1958) function included Brownlie's data and incorporated data for transport of gravels in addition to the sand data used to develop the original Laursen function. The Laursen-Copeland function is very sensitive to the fraction of fine and very fine sand present in the bed. This function is best used when measurements of suspended sediment are available to confirm calculated concentrations of fine material.

The results from these tests are shown in Figures 17 through 20. As expected, different results are obtained for different sediment transport formulae. However, a similar result is obtained for all transport formulae in that sediment deposition occurs in the lower end of the channel and channel outlet. As a result of this deposition, the capacity of the channel is significantly reduced and the peak flow would not be contained by the proposed project. Specific depths of deposition range from 3.0 to 6.7 ft in the channel outlet by the peak of the 100-year flood.

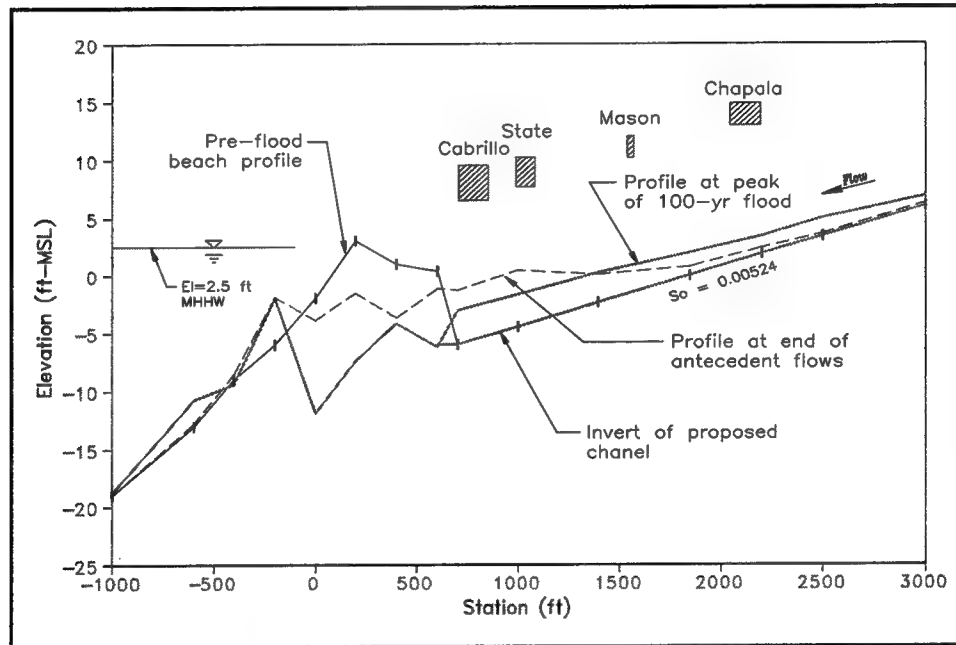


Figure 17. Results of sensitivity analysis at channel outlet for combination of Toffaleti and Schoklitsch functions



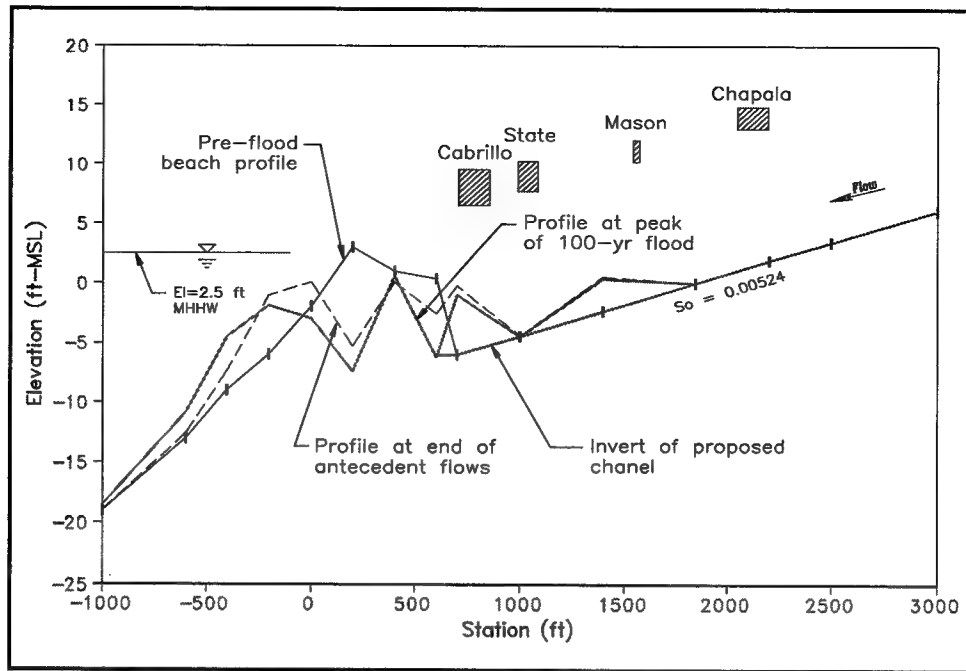


Figure 18. Results of sensitivity analysis at channel outlet for Yang's unit stream power function

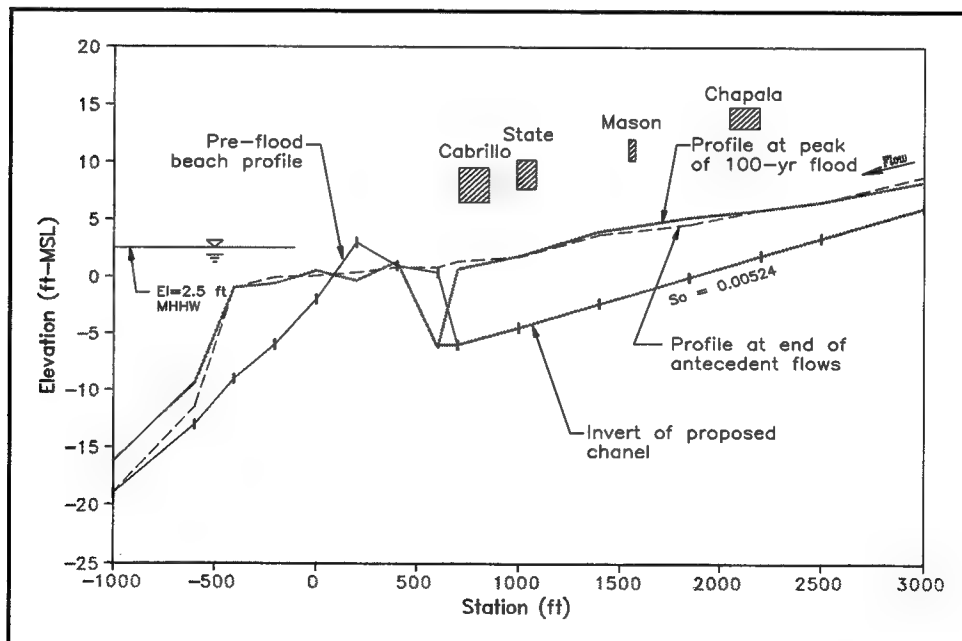


Figure 19. Results of sensitivity analysis at channel outlet for Laursen-Madden function

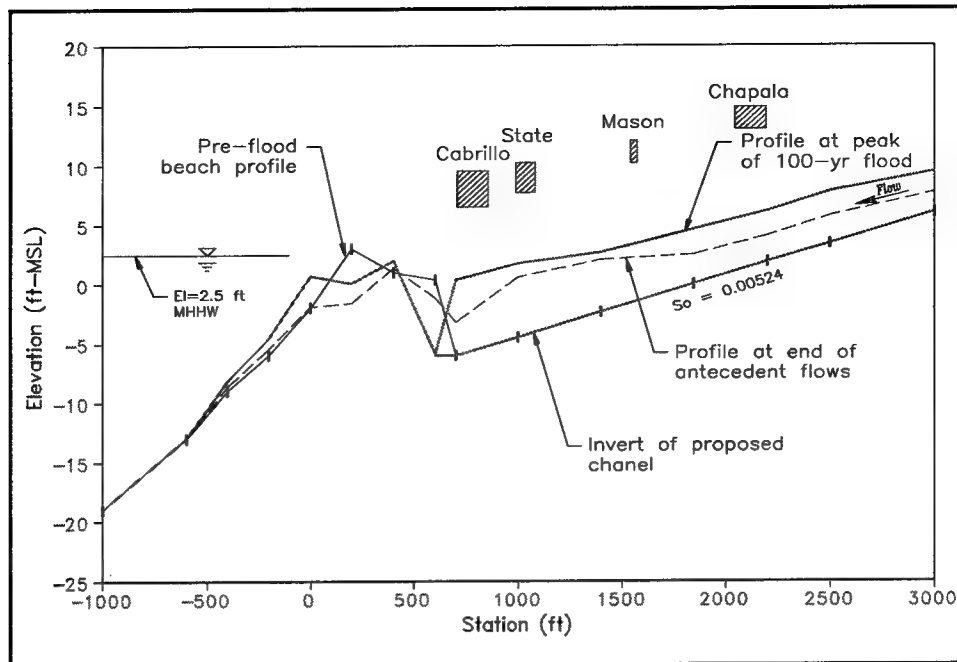


Figure 20. Results of sensitivity analysis at channel outlet for Laursen-Copeland function

**Roughness coefficient.** This sensitivity analysis was performed to evaluate the impact of a low roughness value in the deposition zone of the channel outlet. Specifically, a Manning's  $n$  value of 0.014 was used throughout the proposed concrete-lined channel instead of the 0.022 value used in the tests discussed previously. The combination of the Toffaleti and Meyer-Peter and Müller transport function was used for this test. Results of this test, shown in Figure 21, indicate that deposition would occur to a depth of up to 4.2 ft in the channel outlet by the peak of the 100-year flood. This represents a decrease in sediment depth of 0.8 ft and demonstrates that the depth of sediment deposition in the channel outlet is somewhat sensitive to the roughness value assumed in the deposition zone. However, this test also verifies that even if an extremely low roughness value is assumed near the outlet, significant deposition will occur in the downstream end of the proposed channel and outlet, decreasing the channel capacity to the point that the 100-year flood would not be contained.

**Downstream water surface elevation.** This sensitivity analysis was performed to evaluate the impact of a lower tailwater elevation at the downstream end of the proposed channel. Specifically, a tailwater elevation of 0.0 was used throughout the numerical simulation of the antecedent and 100-year flows instead of the el 2.5 value used in the tests discussed previously. The combination of the Toffaleti and Meyer-Peter and Müller transport function was used for this test. Results of this test shown in Figure 22, indicate that deposition would occur to a depth of up to 3.4 ft in the channel outlet by the peak of the 100-year flood. This represents a decrease in sediment depth of 1.6 ft and

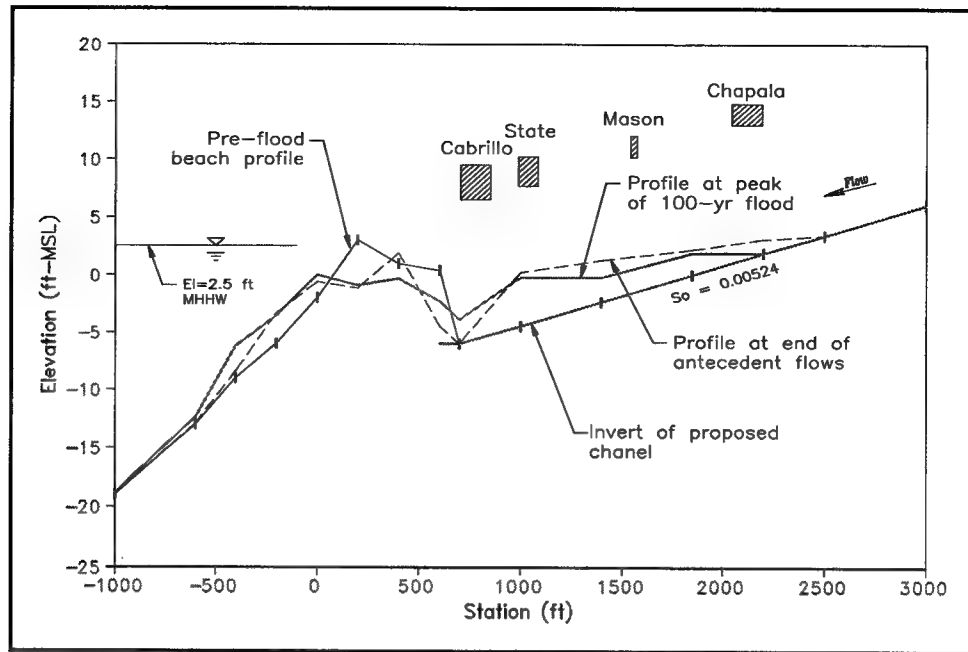


Figure 21. Results of sensitivity analysis for low roughness value in outlet

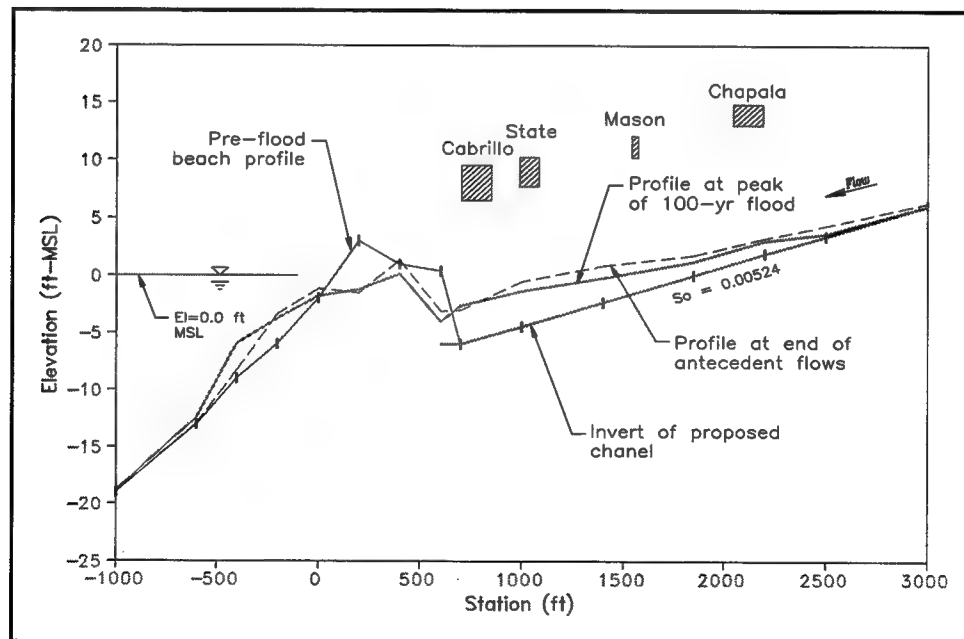


Figure 22. Results of sensitivity analysis for low tailwater elevation

demonstrates that the depth of sediment deposition in the channel outlet is probably more sensitive to the assumed tailwater elevation of the Pacific Ocean than to the roughness value of the channel outlet. However, this test also verifies that significant deposition will occur in the channel outlet, decreasing the channel capacity to the point that the 100-year flood would not be contained.

## **4 Flume Investigations**

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### **Purpose and Scope of Model Investigation**

The purpose of the flume study was to determine the influence of the bed-load transport on hydraulic roughness in the concrete channels proposed for lower Mission Creek and Corte Madera Creek. Specifically, this study was to quantify the increase in the effective hydraulic roughness due to gravel bed-load transport, determine the concentration at which bed forms begin to appear, and identify the characteristics of the bed-load transport.

The flume was also used to study the ability of the design flood to wash out the sand plug that is expected to form at the Mission Creek channel outlet. The flume did not simulate the geometry of the proposed channel, nor was the finer fraction of the bed material simulated. Thus, it would be more difficult for the sand plug to wash out of the flume than in the prototype. If the plug washed out of the flume, the channel would be expected to perform as designed. If, however, the plug did not wash out of the flume, then the experiment would be inconclusive.

### **Description**

The tilting steel flume (Figure 23) used in the study was 80 ft long and 3 ft wide. The flume tests were conducted using steady uniform flow and designed to model the prototype minimum Froude number, slope, and velocity at the flood peak. This was accomplished using a model scale of 1:32.1 based on Froude criteria. Sediment was introduced into the flume by a constant feed sediment hopper (Figure 24). The motor-operated elevator hopper had a 22-cu-ft-capacity and allowed for experiments of about 45 to 60 min.

### **Method of operation**

Before each test, sediment was placed in the hopper and a vibrating rod was used to consolidate the material. The sediment was leveled with the bottom of the flume bed and then lowered approximately 3 to 5 in. below the flume bed. Flow was then introduced into the flume and initial water levels measured without sediment transport. The hopper was then raised at a rate set to achieve

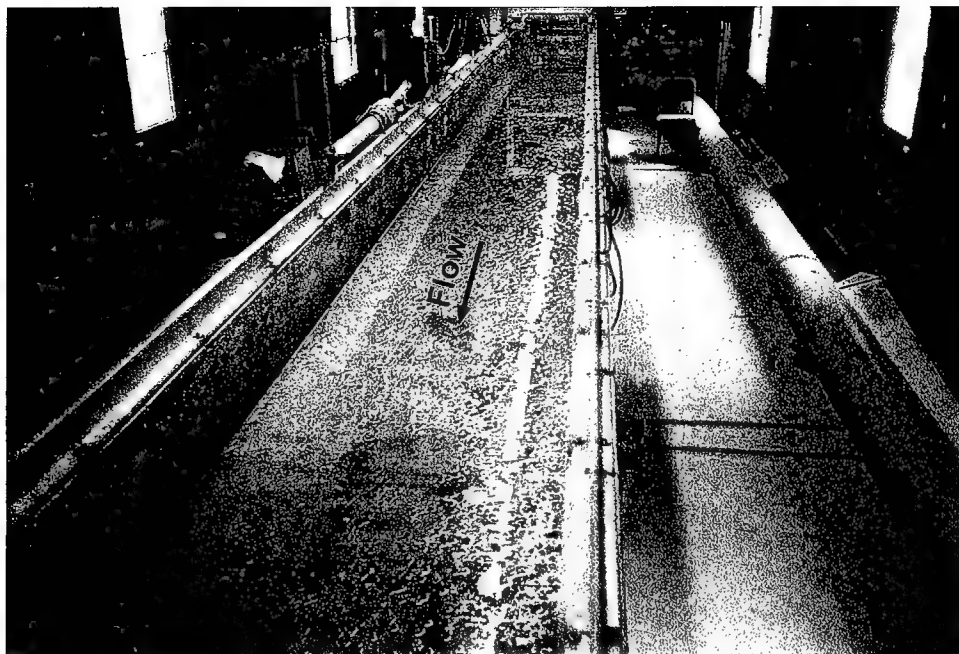


Figure 23. Tilting flume used in study



Figure 24. Sediment feed hopper

specified concentrations. As the hopper rose, sediment was introduced into the flow and eventually a constant feed rate was achieved. Water levels were measured at intervals of 5 to 15 min depending on the sediment feed rate.

Water depths in the flume were measured in five stilling wells. The stilling wells were located on the side of the flume and were connected to the flume sidewalls at a depth of about 0.17 ft. The purpose of the stilling wells was to eliminate the difficulty in determining average depth with the surface waves that are characteristic of rapid flow. The flow exited the flume in free flow.

### Model appurtenances

Flow to the flume was measured using venturi meters. Flow entered the flume from a headbay where a constant head was maintained. Floating and horizontal baffles were used to ensure uniform flow distribution from the headbay.

### Scale relations

The flume study did not attempt to reproduce complete geometric or dynamic similitude. Unit discharge was set in the flume to match the design unit discharge in the prototype according to the accepted equations of hydraulic similitude based on the Froude criteria. Using the available discharge of the flume, this resulted in a model scale of 1:32.1 for Mission Creek and 1:25 for Corte Madera Creek. General relations for transference of flume data to prototype equivalents are listed in the following tabulation. Flume measurements of depth, velocity, and unit discharge can be transferred quantitatively to prototype equivalents by the scale relations. Since the flume did not model the actual geometric shape of the prototype channel, nor the variability in bed slope, similitude for bed shear stress was not achieved. In addition, the boundary roughness in the flume simulated a prototype roughness of 0.016 for Mission Creek and 0.015 for Corte Madera Creek, both of which are higher than the design roughness of 0.014. These differences in base roughnesses of the prototypes and the flume are deemed acceptable because the purpose of the study was to identify the relative increase in hydraulic roughness with increasing bed load.

Characteristic	Dimension <sup>1</sup>	Scale Relations	
		Mission Creek	Corte Madera Creek
Length	$L_r=L$	1:32.1	1:25
Depth	$Y_r=L_r$	1:32.1	1:25
Slope	$S_r=1$	1:1	1:1
Velocity	$V_r=L_r^{1/2}$	1:5.66	1:5
Unit Discharge	$q_r=L_r^{3/2}$	1:181	1:125
Time	$T_r=L_r^{1/2}$	1:5.66	1:5
Manning's coefficient	$n_r=L_r^{1/6}$	1:1.78	1:1.71
<sup>1</sup> Dimensions are in terms of length $L$ .			

In the flume, uniform flow was established and the bed shear stress was determined using the water depth and bed slope (bed, water surface, and total energy line slopes were parallel). The minimum design bed slope at the

commencement of this study, 0.00395 in Mission Creek and 0.0038 in Corte Madera Creek, was tested in the flume. However, in the prototype, distances are insufficient for the establishment of uniform flow, so that prototype friction slopes would always be greater than that tested. In the flume, the aspect ratio was approximately 8 and the sidewalls had negligible effect on the bed-shear stress calculated using the formula  $J_b = (\gamma S)$ , where  $\gamma$  is the specific weight of water and  $y$  is depth. However, the aspect ratio in the prototype is 2.2 for Mission Creek and 3.6 in Corte Madera Creek, and the sidewalls will significantly reduce the bed shear stress (Chow 1959) at a comparable unit discharge. Thus, the increase in prototype bed shear stress due to friction slopes is partially counterbalanced by the decrease in prototype bed shear stress due to sidewall effects. The difference between the bed shear stresses simulated in the flume and the prototype design bed shear stresses were calculated using the procedure outlined by Chow (1959). The minimum calculated energy slope of 0.0056 for Mission Creek, taken from the hydraulic design profiles, was used to calculate bed shear stress in the prototype. The shear stresses in the flume were generally less than the bed shear stress in the prototype with the maximum difference about 7 percent. Bed shear stress calculated using the bed slope and the friction slope in Mission Creek for specified concentrations tested are given in Tables 1-3.

### Roughness coefficients

The average Manning's roughness coefficient  $n$  was determined assuming uniform flow and computed from the measured discharge, friction slope, and hydraulic radius. The Manning's  $n$  value of the flume was determined to be about 0.016 (prototype scale) for Mission Creek and 0.015 for Corte Madera Creek based on clear-water flows. Because the sidewalls of the flume were smoother than the channel bed when bed-load transport occurs, the Manning's  $n$  value of the bed was determined using a sidewall correction method. The procedure used to determine the bed roughness with sidewall correction is outlined by Einstein (1942). It has been determined by Einstein that both bed and sidewall roughness can be expressed by means of Manning's formula:

$$V = \frac{k_c}{n} S^{0.5} R^{0.67} \quad (14)$$

where  $k_c = 1.486$  for non-SI units, 1.0 for SI units

Measuring the depth  $d$  and assuming average roughness for the flume is the roughness for the sidewall (0.0160 or 0.015), and the slope  $S$  and the average velocity  $V$  are constant for all units, the hydraulic radius for the wall  $R_w$  can be determined by using Manning's formula:

$$R_w = \left( \frac{n_w V}{k_c S^{0.5}} \right)^{1.5} \quad (15)$$



<b>Table 1</b> <b>Series A: Bed Shear Stress, <math>J_b = FyS</math></b>		
Specified Concentration ppm	Bed Shear Stress in Flume <sup>1</sup> , $J_b = FyS_o$ , lb/ft <sup>2</sup>	Bed Shear Stress in Prototype <sup>2</sup> , $J_b = 0.75 FyS_o$ , lb/ft <sup>2</sup>
0	2.89	3.08
200	2.89	3.10
400	2.95	3.12
800	3.02	3.18
1,600	3.08	3.22
2,400	3.27	3.33
3,000	3.31	3.34
3,250	3.27	3.41
3,500	3.27	3.41
4,000	3.21	3.20
5,000	3.05	3.26
Note: The flow series of flume experiments are described in Chapter 5. To convert bed shear stress to pascals, multiply by 47.88. $S_o = 0.00395$ and $S_o = S_i$ in the flume. $S_i = 0.0056$ in the prototype which is minimum calculated for the reach with a bed slope of 0.00395. <sup>1</sup> Values are scaled to prototype values. <sup>2</sup> Bed shear stress was reduced to account for sidewall effects for an aspect ratio of 2.2 using Chow's shear stress distribution curves (Chow 1959, pp 168-170).		

<b>Table 2</b> <b>Series B: Bed Shear Stress, <math>J_b = FyS</math></b>		
Specified Concentration ppm	Bed Shear Stress in Flume <sup>1</sup> , $J_b = FyS_o$ , lb/ft <sup>2</sup>	Bed Shear Stress in Prototype <sup>2</sup> , $J_b = 0.75 FyS_o$ , lb/ft <sup>2</sup>
0	2.89	3.08
200	2.89	3.08
400	2.92	3.10
800	3.05	3.18
1,600	3.05	3.17
2,400	3.14	3.31
2,700	3.24	3.35
3,000	3.40	3.37
Note: The flow series of flume experiments are described in Chapter 5. To convert bed shear stress to pascals, multiply by 47.88. $S_o = 0.00395$ and $S_o = S_i$ in the flume. $S_i = 0.0056$ in the prototype which is minimum calculated for the reach with a bed slope of 0.00395. <sup>1</sup> Values are scaled to prototype values. <sup>2</sup> Bed shear stress was reduced to account for sidewall effects for an aspect ratio of 2.2 using Chow's shear stress distribution curves (Chow 1959, pp 168-170).		

**Table 3**  
**Series C: Bed Shear Stress,  $J_b = FyS$**

Specified Concentration ppm	Bed Shear Stress in Flume <sup>1</sup> , $J_b = FyS_o$ , lb/ft <sup>2</sup>	Bed Shear Stress in Prototype <sup>2</sup> , $J_b = 0.75 FyS_o$ , lb/ft <sup>2</sup>
0	2.89	3.08
500	2.92	3.11
1,000	3.05	3.17
2,000	3.11	3.27
3,000	3.27	3.42

Note: The flow series of flume experiments are described in Chapter 5. To convert bed shear stress to pascals, multiply by 47.88.  $S_o = 0.00395$  and  $S_o = S_f$  in the flume.  $S_f = 0.0056$  in the prototype which is minimum calculated for the reach with a bed slope of 0.00395.  
<sup>1</sup> Values are scaled to prototype values.  
<sup>2</sup> Bed shear stress was reduced to account for sidewall effects for an aspect ratio of 2.2 using Chow's shear stress distribution curves (Chow 1959, pp 168-170).

The area associated with the wall is

$$A_w = 2 d R_w \quad (16)$$

Since

$$A = A_w + A_b \quad (17)$$

the area associated with the bed is

$$A_b = d (b - 2 R_w) \quad (18)$$

where  $b$  = base width

and the corresponding hydraulic radius associated with the bed is

$$R_b = d \left( 1 - 2 \frac{R_w}{b} \right) \quad (19)$$

Finally, by using the radius  $R_b$ , the bed roughness factor  $n_b$  is obtained

$$n_b = \frac{k_c}{V} S^{0.5} R_b^{0.67} \quad (20)$$

Thus the roughness associated with the bed,  $n_b$ , can be calculated from measured values of velocity, slope, depth, base width, and  $n_w$  using the following combined equation:

$$n_b = \frac{k_c S^{0.5}}{V} \left\{ d \left[ 1 - \frac{2}{b} \left( \frac{n_w V}{k_c S^{0.5}} \right)^{1.5} \right] \right\}^{0.67} \quad (21)$$

To account for differences in the simulated flume and design prototype roughnesses, the following procedure was adopted. The total bed hydraulic roughness can be expressed as:

$$n_{bed} = n_{boundary} + n_{bed\ load} \quad (22)$$

where

$n_{bed}$  = total bed roughness due to channel bed and bed load roughnesses

$n_{boundary}$  = roughness due to the channel bed with no bed load movement

$n_{bed\ load}$  = roughness due to bed-load movement.

Solving for the roughness due to bed-load movement, Equation 22 can be written:

$$n_{bed\ load} = n_{bed} - n_{boundary} \quad (23)$$

The model ratios of 1:32.1 and 1:25 were used to convert the  $n_{bed\ load}$  from the flume to the prototype as follows:

For Mission Creek

$$n_r = L_r^{\frac{1}{6}} = \left( \frac{1}{32.1} \right)^{\frac{1}{6}} = \frac{1}{1.783} \quad (24)$$

$$n_{p\ bed\ load} = 1.783\ n_{m\ bed\ load} \quad (25)$$

For Corte Madera Creek

$$n_r = L_r^{\frac{1}{6}} = \left( \frac{1}{25} \right)^{\frac{1}{6}} = \frac{1}{1.710} \quad (26)$$

$$n_{p\ bed\ load} = 1.710\ n_{m\ bed\ load} \quad (27)$$

where

$n_r$  = model ratio for Manning's  $n$ -value,  $n_r = n_m/n_p$

$n_p$  =  $n$  value of prototype

$n_m = n$  value of flume

$L_r =$  model ratio

Total bed roughness coefficients are reported herein using a prototype boundary roughness of 0.016. However, calculated model and prototype bed-load roughnesses are also tabulated so that total prototype bed roughness can be determined for different boundary roughnesses. For example, for Mission Creek, if the bed Manning's  $n$  value of the prototype concrete-lined channel is assumed to be 0.014 with clear-water flows, then the total hydraulic roughness neglecting sidewall effects and including bed-load movement in the prototype is:

$$n_p = 0.014 + 1.783n_{m \text{ bed load}} \quad (28)$$

## 5 Flume Experimental Results

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### Mission Creek

Four series of flume experiments were conducted for the Mission Creek study. Series A was conducted using a uniform grain size that simulated the large cobbles found in the armor layer upstream from the proposed project (216 mm,  $F = 1.4$ ); Series B was conducted using a uniform grain size that simulated the prototype  $d_{84}$  (108 mm,  $F = 1.4$ ); Series C was conducted using a gradation that simulated the gravel portion of the creek bed; and Series D was conducted to study washout of material deposited at the downstream end of the concrete channel at its confluence with the ocean. The bed-load feed rate range for each series varied from 200 ppm up to 5,000 ppm. The maximum estimated bed-load concentration from the numerical model study was 1,300 ppm.

The bed material gradation used for Series C, simulated the coarser 75 percent of the prototype gradation. Size and weight of fine sand in the prototype cannot be simulated in the model without introducing error due to the cohesive forces, characteristic of silts and clays; therefore exact reproduction of the finer portion of the prototype bed material (25 percent) was not attempted. However, this was not considered critical because this class of material should behave as wash load in both the model and prototype. Model and prototype bed gradation is compared in Figure 25.

Water depths were used to determine Manning's roughness coefficient, average and maximum shear stress, and Froude number. Actual sediment concentration was lower than the specified concentration initially until uniform feed was established at the hopper. The Manning's roughness coefficient determined at the uniform feed rate was used to calculate the Manning's bed-load roughness.

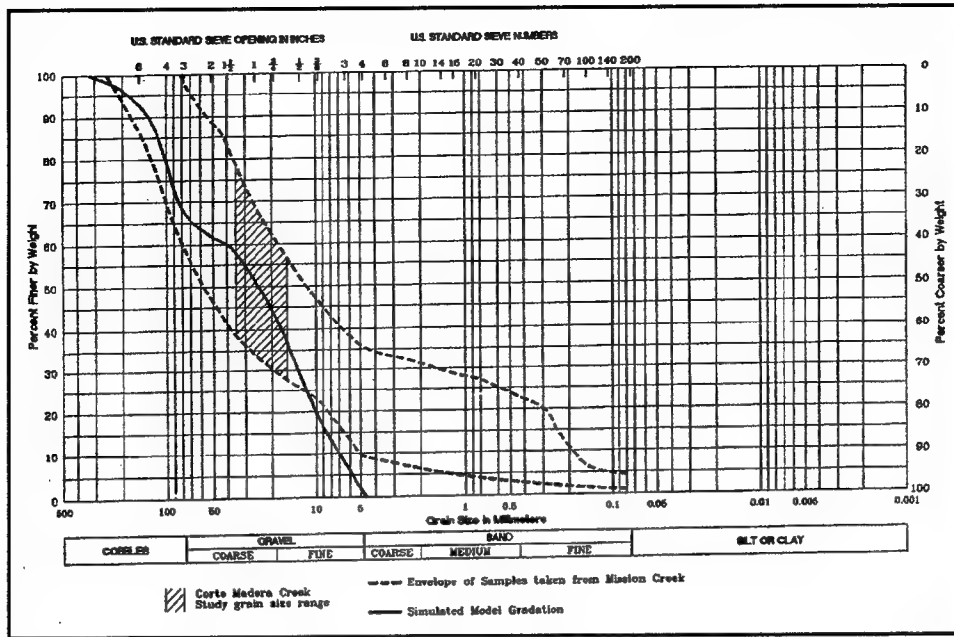


Figure 25. Comparison of model and prototype bed gradation

Series A - C were conducted to determine the increase in bed roughness with increasing bed-load concentration. As long as the bed-load particles remained in motion along the flume bed, a consistent relationship was apparent. When the bed-load feed rate exceeded the transport capacity of the flume, deposits began to occur in the flume so that the feed rate and bed-load concentration were no longer the same. At this point bed roughness and bed-load concentration began to vary along the length of the flume. It was the purpose of the flume study to identify this threshold, but not to determine bed roughness for the fully covered gravel bed, nor for the unsteady, nonuniform transition bed.

#### Series A: $d_{\max}$ material

The calculated roughness coefficient for each specified sediment concentration tested during Series A was plotted versus time, as shown in Plates 1-11. The bed-load feed rate for the  $d_{\max}$  material varied from 200 ppm to 5,000 ppm. For bed-load feed rates up to 3,000 ppm, the roughness coefficient increased from the no-sediment concentration value of 0.016 (prototype) to a constant value as flow conditions equalized (Plates 1-7). At bed-load feed rates above 3,000 ppm, the roughness coefficient initially increased to approximately the value for 3,000 ppm and then decreased, as the sediment transport capacity of the flow was exceeded and bed forms began to develop (Plates 8-11). The relationship between the equilibrium roughness coefficient and bed-load concentration for the Series A experiments is shown in Plate 12. The relation between Manning's bed roughness coefficient and specified sediment concentration for Series A, B, and C is shown in Figure 26, which also shows the expected range of gravel concentrations.

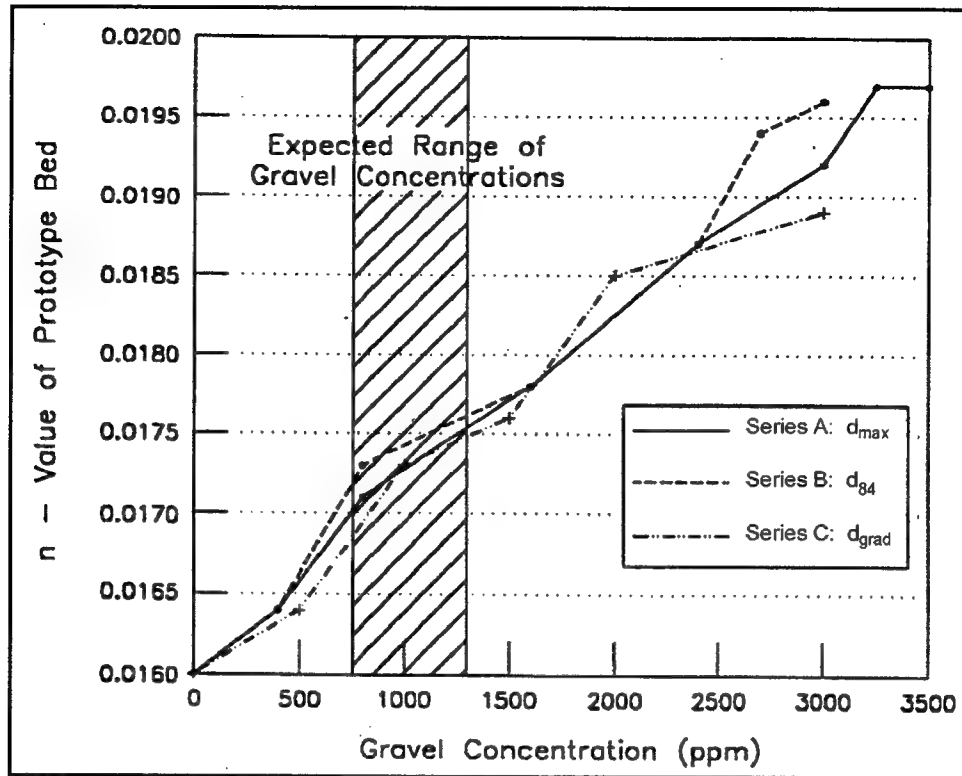


Figure 26. Relation between Manning's bed roughness coefficient and specified gravel concentrations for Mission Creek

For bed-load concentrations up to 3,000 ppm, the sediment was transported downstream in a bouncing and rolling motion. The sediment accumulated in small clusters, shown in Figure 27, as it moved downstream until the turbulence of the flow caused them to disperse. The sediment moved along the bed of the flume at velocities slightly less than the velocity of water, creating a drag force that increased the effective hydraulic roughness. An increase in the hydraulic roughness was first detected at a sediment concentration of 200 ppm with an increase of approximately 1.2 percent above that for clear-water flow. The maximum Manning's roughness coefficient for the flume bed at a sediment concentration of 3,000 ppm was 0.0192 (prototype), an increase from the no-sediment value of approximately 20 percent. The final Manning's roughness coefficient for each specified concentration for Series A is given in Table 4.

For bed-load feed rates of 3,250 ppm and greater, bed forms developed. During the experiment with a bed-load feed rate of 3,250 ppm, the bed load traveled downstream in large clusters at a slower velocity than the flow. These clusters increased significantly in volume until bed forms developed. The bed forms appeared approximately 45 ft (model) from the hopper and slowly moved upstream until crossbars developed. Figure 28 shows the crossbars in the flume for the bed-load feed rate of 3,500 ppm. Because the flow was not capable of transporting the bed load downstream at bed-load feed rates of 3,500, 4,000, and 5,000 ppm, aggradation occurred downstream of the hopper. During experiments with bed-load feed rates of 3,250 and 3,500 ppm, the hydraulic roughness never

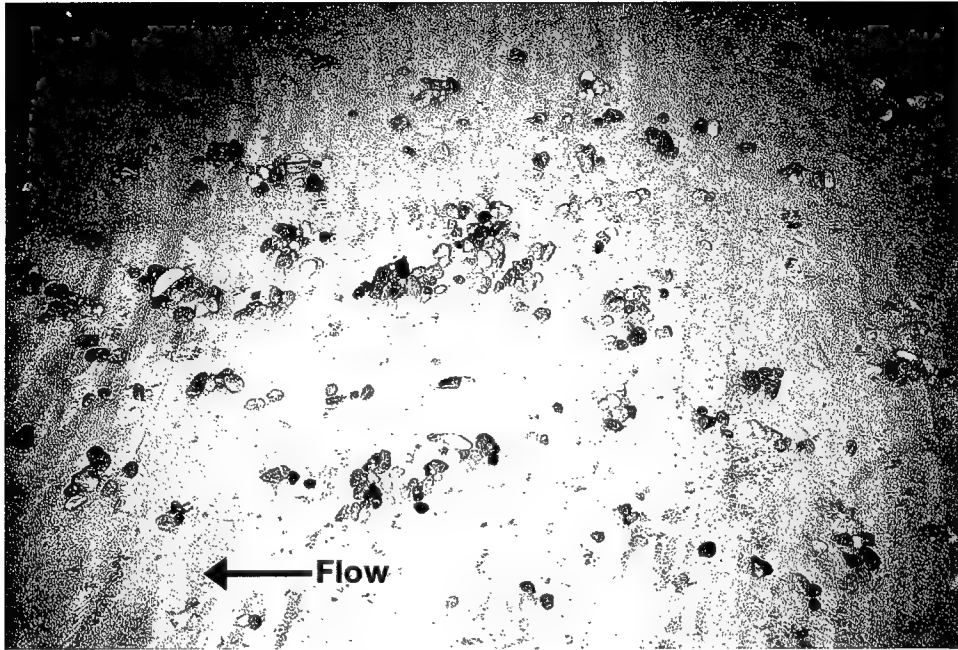


Figure 27. Accumulation of  $d_{\max}$  sediment clusters



Figure 28. Crossbars that developed during Series A experiment at specified sediment concentration of 3,500 ppm

stabilized. The Manning's roughness coefficient peaked shortly after the tests began at 0.0197 and then steadily decreased (Plates 8 and 9). During experiments with bed-load feed rates of 4,000 and 5,000 ppm, the roughness coefficient



Table 4 Series A: $d_{max}$ Data Analysis									
Bed-Load Feed Rate ppm	Depth, in Flume ft	Velocity, in Flume ft/sec	Froude Number	Flume			Prototype		% Increase
				Average Manning's Roughness $n$	Bed Manning's Roughness $n$	Average Manning's Roughness $n$	Bed Manning's Roughness $n$		
0	0.366	4.58	1.33	0.0090	0.0090	0.0160	0.0160	0.0000	0.0
200	0.369	4.52	1.31	0.0091	0.0091	0.0162	0.0162	0.0002	1.2
400	0.372	4.48	1.31	0.0092	0.0092	0.0164	0.0165	0.0005	3.1
800	0.378	4.41	1.28	0.0095	0.0096	0.0169	0.0173	0.0013	8.1
1,600	0.383	4.35	1.24	0.0098	0.0100	0.0175	0.0176	0.0016	10.0
2,400	0.396	4.21	1.21	0.0102	0.0105	0.0182	0.0190	0.0030	18.7
3,000	0.397	4.20	1.17	0.0105	0.0108	0.0187	0.0192	0.0032	20.0
3,250 <sup>1</sup>	0.406	4.10	1.13	0.0107	0.0111	0.0191	0.0197	0.0037	23.1
3,500 <sup>1</sup>	0.406	4.10	1.18	0.0107	0.0111	0.0191	0.0197	0.0037	23.1
Note: The four series of flume experiments are described in Chapter 5. <sup>1</sup> Bed-load concentrations in which bed forms developed.									

peaked at 0.0197 and 0.0186, respectively, and then decreased but eventually stabilized at 0.0179 and 0.0178, respectively (Plates 10 and 11). Actual bed-load concentrations for these conditions were not determined.

The Froude number decreased from 1.33 for no-sediment transport to 1.13 for bed-load feed rate of 3,250 ppm. The Froude number for each bed-load concentration for Series A is given in Table 1. At bed-load feed rates of 3,500, 4,000, and 5,000 ppm, the Froude number increased to 1.18, 1.23, and 1.21, respectively.

### **Series B: $d_{84}$ material**

Experiments for the  $d_{84}$  material, Series B, were conducted with feed rates varying from 200 to 3,000 ppm. Plots of the roughness coefficient for each bed load concentration versus time are shown in Plates 13-20. For bed-load feed rates up to 2,700 ppm, the roughness coefficient increased from the clear-water flow value of 0.016 to a constant value as flow conditions equalized (Plates 13-19). For the bed-load feed rate of 3,000 ppm, bed forms developed and the hydraulic roughness never stabilized (Plate 20). The relation between Manning's bed roughness coefficient and bed-load concentration for Series B is shown in Plate 21 and Figure 26.

Similar to the  $d_{max}$  experiments, the  $d_{84}$  material moved down the flume in bouncing or rolling motions at velocities slightly less than the velocity of the water, creating a drag force that increased the effective hydraulic roughness. Small clusters formed as the sediment moved downstream until the force of the flow caused them to disperse. The maximum Manning's roughness coefficient for Series B measured at a bed-load concentration of 2,700 ppm, an increase of 21 percent from the no-sediment value. At the lowest bed-load concentration, 200 ppm, the effective hydraulic roughness increased approximately 1.2 percent. The final Manning's roughness coefficient for each bed-load concentration for Series B is given in Table 5.

For the bed-load feed rate of 3,000 ppm, large clusters accumulated as the sediment traveled downstream until bed forms developed. As with Series A, the bed forms were first observed approximately 45 ft (model) from the hopper and slowly moved upstream until crossbars developed. The maximum roughness coefficient measured during this experiment was 0.0196. After the hydraulic roughness peaked, it decreased to a value of 0.0178 and then began to increase again. The Manning's roughness coefficient at the end of the experiment was measured at 0.0193.

The Froude number for each specified concentration for Series B is shown in Table 5. The Froude number for the no-sediment transport was 1.33 and decreased approximately 14.3 percent to 1.13 for the bed load feed rate of 3,000 ppm.

**Table 5**  
**Series B:  $d_{84}$  Data Analysis**

Bed-Load Feed Rate ppm	Depth, In Flume ft	Velocity, In Flume ft/sec	Froude Number	Flume			Prototype			% Increase
				Average Manning's Roughness $n$	Bed Manning's Roughness $n$		Average Manning's Roughness $n$	Bed Manning's Roughness $n$	Increase In Roughness due to Bed Load	
0	0.366	4.58	1.33	0.0090	0.0090		0.0160	0.0160	0.0000	0.0
200	0.367	4.54	1.32	0.0091	0.0091		0.0162	0.0162	0.0002	1.2
400	0.368	4.53	1.30	0.0092	0.0092		0.0164	0.0164	0.0004	2.5
800	0.378	4.41	1.26	0.0096	0.0097		0.0171	0.0173	0.0013	8.1
1,600	0.377	4.42	1.24	0.0098	0.0100		0.0175	0.0178	0.0018	11.2
2,400	0.393	4.24	1.19	0.0102	0.0105		0.0182	0.0187	0.0027	16.9
2,700	0.399	4.18	1.16	0.0105	0.0109		0.0187	0.0194	0.0034	21.2
3,000 <sup>1</sup>	0.405	4.11	1.13	0.0106	0.0110		0.0189	0.0196	0.0036	22.5

<sup>1</sup> Bed-load concentrations in which bed forms developed.

<sup>1</sup> Bed-load concentrations in which bed forms developed.

### **Series C: graded material**

Experiments for the graded material, Series C, were conducted with concentrations varying from 500 ppm to 3,000 ppm. The gradation of the bed material simulated the gravel portion of the creek bed. The gradation curves showing the envelope for samples taken at Mission Creek and the model material used in the experiments are shown in Figure 25. The roughness coefficient for each bed-load concentration was plotted versus time in Plates 22-27. For each bed-load concentration, the roughness coefficient increased from the clear-water flow value of 0.016 to a constant value as the flow conditions equalized. No bed forms were observed during Series C. Plate 28 and Figure 26 show the relation between Manning's roughness coefficient and bed-load concentration for Series C.

For each bed-load concentration, it was observed that the gravel moved along the bottom of the flume bed in a bouncing and rolling motion, and the sand moved in a sliding motion. Both materials moved at slower velocities than the flow, causing a drag force that increased the effective hydraulic roughness. The maximum Manning's roughness coefficient for the flume prototype bed at a bed-load concentration of 3,000 ppm was 0.0189, an 18 percent increase from the no-sediment value. The final Manning's roughness coefficient for each specified concentration for Series C is given in Table 6. An increase in the hydraulic roughness of approximately 3.1 percent was first detected while testing the specified concentration of 500 ppm. A maximum threshold concentration for the graded material was not determined.

The Froude number decreased from 1.33 for no-sediment transport to 1.17 for a bed-load concentration of 3,000 ppm, a decrease of approximately 12 percent. The Froude number for each specified concentration for Series C is given in Table 6.

### **Series D: simulation of channel outlet**

The tilting flume was used to study the ability of the proposed channel to clear the sand plug at the channel outlet during the 1 percent change exceedance flood. A sand plug forms naturally in the existing channel during periods of low or negligible flow and is expected to form in the design channel where the channel invert is below mean sea level. This experiment was qualitative in nature since the flume did not reproduce the geometry of the proposed channel nor the finer fractions of the material deposited in the downstream end of the channel. Additionally, the gradation of the material representing the beach was coarser than the prototype beach material because the fine sand fractions could not be scaled without introducing cohesive bonding effects in the flume. Unit discharge, minimum slope, velocity, and tailwater of the prototype channel were scaled according to Froude criteria.

The discharge in the model was varied to simulate the first 20 hr of the 1 percent chance exceedance hydrograph. Antecedent flows were not modeled, but were included in the simulation by setting the initial bed profile equal to the

**Table 6**  
**Series C: Graded Material Data Analysis**

Bed-Load Feed Rate ppm	Depth, In Flume ft	Velocity, In Flume ft/sec	Froude Number	Flume			Prototype			% Increase
				Average Manning's Roughness <i>n</i>	Bed Manning's Roughness <i>n</i>	Average Manning's Roughness <i>n</i>	Average Manning's Roughness <i>n</i>	Bed Manning's Roughness <i>n</i>	Increase In Roughness due to Bed Load	
0	0.366	4.55	1.33	0.0090	0.0090	0.0160	0.0160	0.0000	0.0	
500	0.370	4.50	1.30	0.0092	0.0093	0.0165	0.0166	0.0006	3.7	
1,000	0.377	4.42	1.27	0.0096	0.0097	0.0171	0.0173	0.0013	8.1	
1,500	0.389	4.28	1.24	0.0097	0.0099	0.0173	0.0176	0.0016	10.0	
2,000	0.396	4.21	1.20	0.0101	0.0104	0.0180	0.0185	0.0025	15.6	
3,000	0.407	4.09	1.17	0.0103	0.0106	0.0184	0.0189	0.0029	18.1	
Note: Bed forms developed during Series C.										

Note: Bed forms developed during Series C.

observed thalweg profile of the existing outlet, estuary, and beach berm. Figure 29 is a profile of the channel and its sand plug at the outlet and Figures 30 and 31 show the channel bed and beach berm in the flume. Inflow sediment concentration was 1,300 ppm and was based on values computed by the HEC-6W numerical modeling.

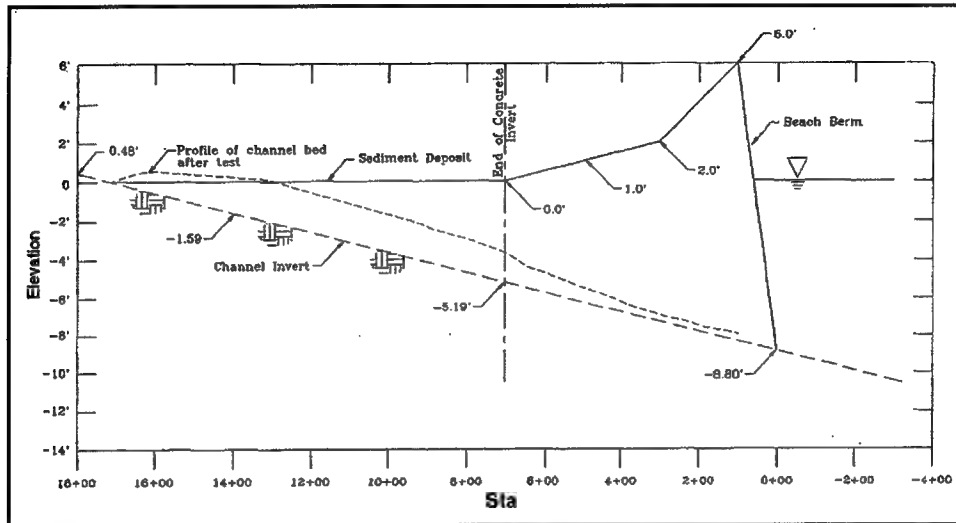


Figure 29. Profile of channel bed and its sand plug at the outlet before and after Series D experiment

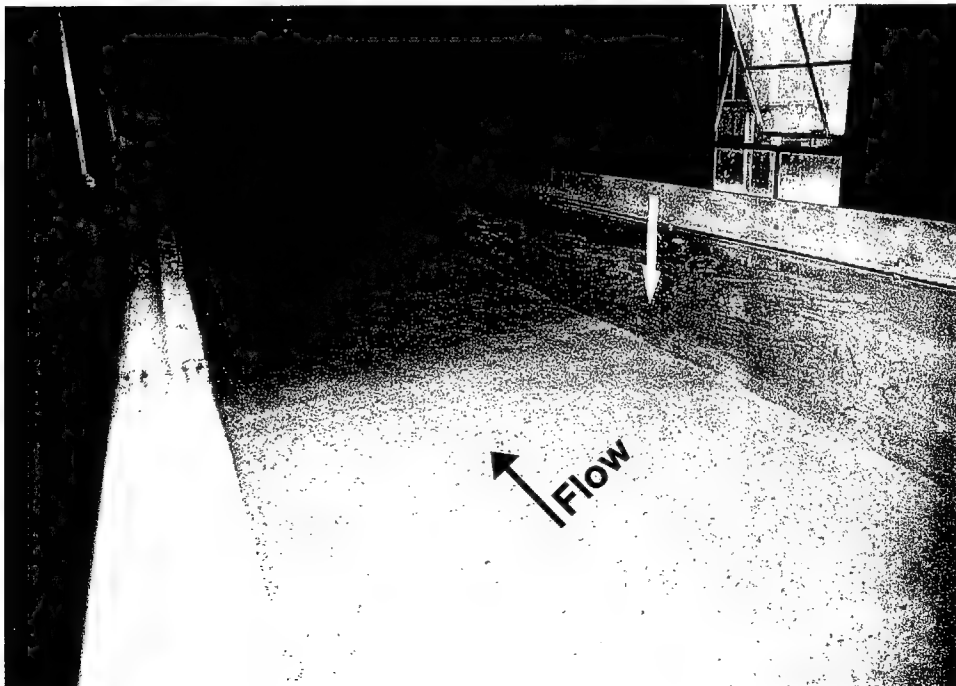


Figure 30. Channel bed profile. Arrow indicates end of concrete channel

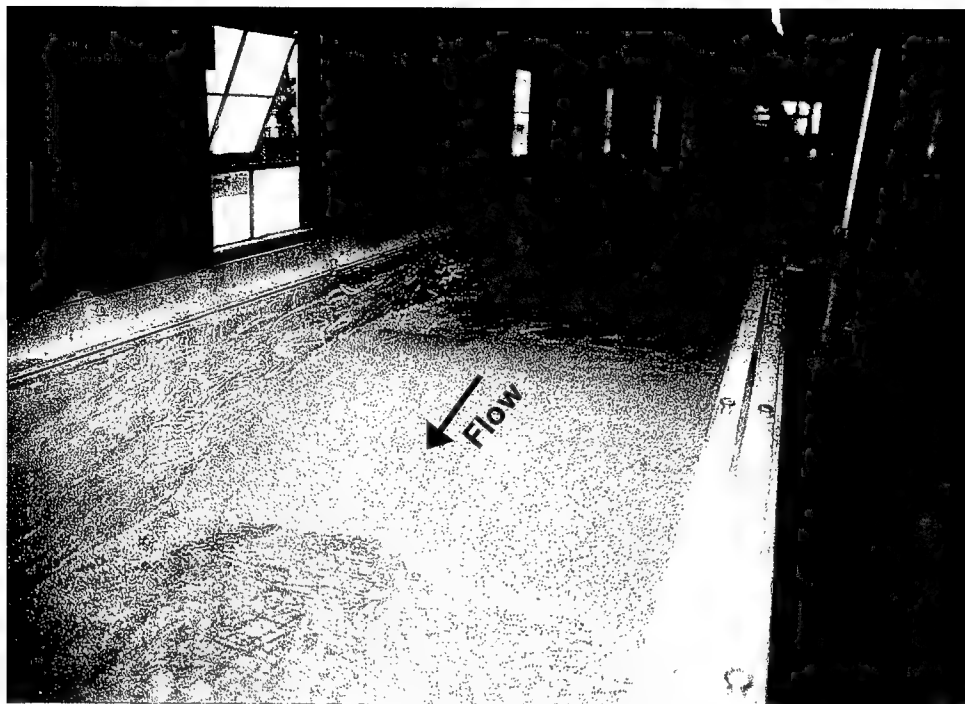


Figure 31. Beach berm at channel outlet

Recognizing the limitation imposed because the material in the flume was coarser than in the prototype, it was reasoned that if the coarser deposit was removed before the flood peak in the model, it could be concluded that the deposit with its finer bed material would be removed in the prototype. However, the experiments indicated the sand plug would not be cleared by the peak of the 1 percent chance exceedance flood. The result of these tests neither verified the results of the numerical simulation nor did it contradict the computed results.

In summary, the results from the flume study of the outlet were inconclusive because the sediment gradations of the bed material at the beach as well as the suspended material were not accurately scaled.

## Corte Madera Creek

The Corte Madera Creek study was conducted for the U. S. Army Engineer District, Sacramento. These experiments used a grain size that simulated the prototype range of 16 mm-32 mm, which represented the coarser 5 to 25 percent of bed material found in Corte Madera Creek. This range is plotted in Figure 25, which shows that it simulates the coarser 40 to 60 percent of Mission Creek. The slope of the flume was 0.0038. The Corte Madera study also had varied discharges of 3,500 cfs, 5,000 cfs, 6,000 cfs, and 6,900 cfs for each range of bed-load concentrations (1,000 ppm, 2,000 ppm, and 3,000 ppm). The results of this study are provided in Table 7.

Table 7 Corte Madera Creek Data Analysis									
Bed-Load Feed Rate ppm	Depth, in Flume ft	Velocity, in Flume ft/sec	Froude Number	Flume			Prototype		
				Average Manning's Roughness <i>n</i>	Bed Manning's Roughness <i>n</i>	Average Manning's Roughness <i>n</i>	Bed Manning's Roughness <i>n</i>	Increase in Roughness due to Bed Load	% Increase
Discharge: 3,500 cfs									
0	0.238	3.56	1.27	0.0090	0.0090	0.0154	0.0154	0.0000	0.0
1,000	0.244	3.47	1.22	0.0093	0.0094	0.0159	0.0160	0.0006	3.9
2,000	0.253	3.34	1.16	0.0099	0.0100	0.0169	0.0172	0.0018	11.7
Discharge: 5,000 cfs									
0	0.302	4.01	1.28	0.0090	0.0090	0.0154	0.0154	0.0000	0.0
1,000	0.310	3.90	1.23	0.0095	0.0096	0.0162	0.0164	0.0010	6.5
2,000	0.318	3.80	1.17	0.0099	0.0101	0.0169	0.0172	0.0018	11.7
Discharge: 6,000 cfs									
0	0.338	4.30	1.30	0.0090	0.0090	0.0154	0.0154	0.0000	0.0
1,000	0.346	4.20	1.25	0.0094	0.0094	0.0161	0.0161	0.0007	4.5
2,000	0.359	4.05	1.18	0.0099	0.0101	0.0169	0.0177	0.0019	12.3
Discharge: 6,900 cfs									
0	0.369	4.53	1.31	0.0090	0.0090	0.0154	0.0154	0.0000	0.0
1,000	0.377	4.43	1.27	0.0093	0.0094	0.0159	0.0160	0.0006	3.9
2,000	0.391	4.27	1.20	0.0098	0.0100	0.0168	0.0172	0.0018	11.7
3,000	0.410	4.08	1.10	0.0105	0.0109	0.0180	0.0187	0.0033	21.4
Note: Corte Madera Creek flume study was conducted using steady uniform flow at a model scale of 1:25 based on Froude criteria.									



For each bed-load concentration, the sediment moved along the bottom of the flume bed in a sliding motion similar to Series C (graded material) for Mission Creek. The maximum Manning's roughness coefficient measured for the flume bed was 0.0187 (prototype scale) with the discharge of 6,900 cfs and a bed-load concentration of 3,000 ppm. This represented an increase of approximately 21 percent. For the bed-load concentration of 2,000 ppm, the bed roughness coefficient was measured at 0.0172 or 0.0173 for simulated discharge ranges between 3,500 and 6900 cfs. This represented an increase in roughness of about 12 percent. The bed roughness coefficient for the bed-load concentration of 1,000 ppm ranged between 0.0160 and 0.0164, an increase of between 4 and 6 percent. The maximum threshold concentration for Corte Madera Creek was not determined. The Froude number increased with the increase of discharge. However, it decreased approximately 8.7 percent with no-sediment transport to a bed-load concentration of 2,000 ppm. The Froude number for each bed-load concentration for Corte Madera Creek is given in Table 7.

The results of this study are plotted along with the Mission Creek results in Figure 32. The data obtained from the Mission Creek study, which simulated 216 mm, 108 mm, and a gradation of 4.8 mm to 216 mm, plotted very close to the data from the Corte Madera Creek study, which simulated 22 mm. A simple linear regression line through the data indicates the gravel bed-load contribution to the Manning's roughness coefficient can be determined by the following equation with an  $R^2$  of 0.93:

$$n_{bed\ load} = 1.0312 \times 10^{-6} C_{gravel} \quad (29)$$

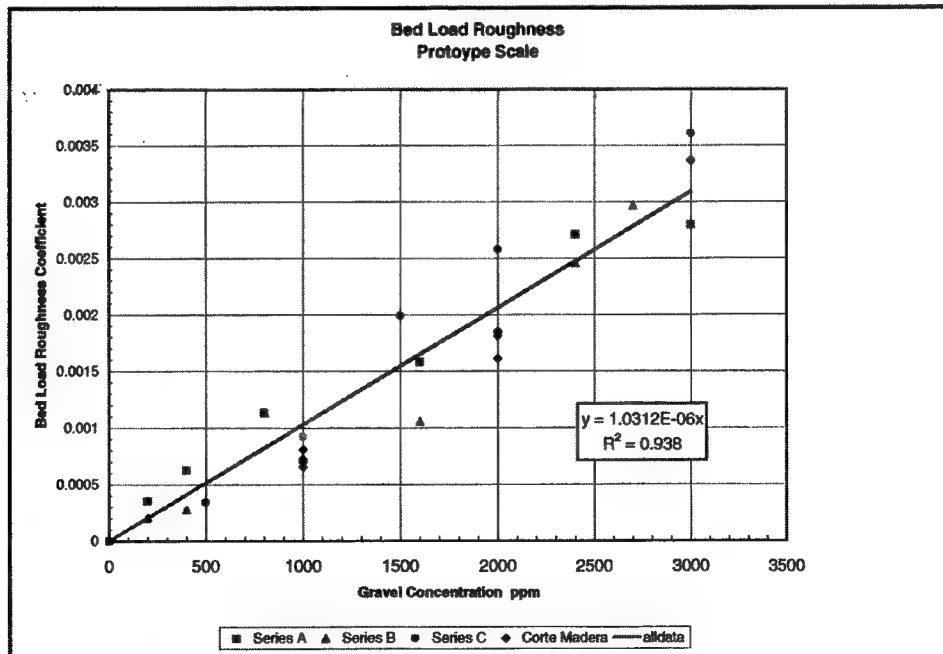


Figure 32. Mission Creek and Corte Madera Creek data results

Part of the study on Corte Madera Creek included simulating a hydraulic jump in the flume to see if bed forms developed downstream from the jump. This was accomplished by raising the tailgate so the jump would occur approximately 60 ft downstream from the hopper. These experiments were visual only and were conducted for bed-load concentrations of 500 and 1,000 ppm, for each of the previously stated discharges. At the beginning of each experiment, the hydraulic jump did not seem to affect the sediment transport; however, as the increased sediment was introduced into the flow, bed forms began to develop at the jump and the jump slowly moved upstream. This event occurred for each experiment. It was not determined at what bed-load concentration the hydraulic jump would not affect sediment transportation. At the end of each experiment, the hydraulic jump was located approximately 40 ft downstream from the hopper.

## 6 Conclusions and Recommendations

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### Conclusions

Detailed sedimentation analyses have been conducted for the proposed channel improvement for lower Mission Creek. The results of the sedimentation analysis indicate that sediment would deposit in the downstream end of the channel prior to and during the 1 percent chance exceedance flood decreasing the channel capacity so that it could not contain the peak flow rate.

Problems with the proposed design are due to a combination of factors: (a) the channel is required to exit onto the beach at the relatively low elevation of -6.0 for the channel to convey flood flows under the low chords of the State Street and Cabrillo Boulevard bridges; and (b) a debris basin is not included as part of the proposed channel design. Hence, it is possible that large volumes of sediment may enter the channel and plug the channel outlet.

Additional flume tests were conducted to investigate the ability of the channel outlet to clear the sand plug during the 1 percent chance exceedance flood. The results from these tests were inconclusive but provided some qualitative insight to the results obtained from the numerical analysis of the channel outlet.

Predicted bed-load transport rates from the numerical model have been coupled with measured hydraulic roughnesses from the flume study to determine the effect of the gravel bed load for the Mission Creek Project. Based on the numerical modeling completed by the Los Angeles District, the maximum concentration of bed-load material flowing into the proposed concrete-lined channel will be about 1,300 ppm. Results from the flume study indicate this bed-load concentration will increase the hydraulic roughness of the bed from 0.014 to 0.0153. When combined with side wall effects, a composite design  $n$  value of 0.015 is computed. Additionally, results from the flume study reveal that bed forms do not form under project conditions until gravel concentrations exceed 2,700 ppm.

Based on the results of these analyses, it can be concluded that the proposed project should not be constructed unless the debris problem is resolved. The alternative solutions to the debris problem are beyond the scope of this report and the analyses conducted during the PED phase of the study. However, some general recommendations regarding further study of this project are discussed in the following section.

## Recommendations

In general, additional study is required to develop a workable flood-control project for lower Mission Creek. If the use of a concrete-lined channel is analyzed in future studies, it appears that the factors most important to the viability of the design are the debris load and the invert elevation at the outlet.

Positive debris control upstream of the deposition zone of the channel should be considered to decrease the impact of sediment deposition near the outlet. This positive debris control could conceivably consist of debris basins, debris traps, and/or other means that reduce the volume of debris flowing into lower Mission Creek. The HEC-6W (HEC-6) numerical models developed for these analyses can be modified and used to study project alternatives that include debris control. Specifically, the models can be used to estimate the sediment volumes by size fraction inflowing the project reach and the impact of any proposed debris basins or sediment traps on the sedimentation processes of lower Mission Creek.

Additional analysis of the invert elevation at the outlet is recommended. In general, the higher the invert elevation, the less deposition would occur near the outlet since the effect of backwater from the ocean would be reduced. One recommendation might include locating the invert elevation roughly equal to the existing thalweg elevation. This alternative requires significant study and may decrease (or eliminate) the deposition problem in this area at the cost of decreased conveyance under the State Street and Cabrillo Boulevard bridges.

Since adjustment and circumstantiation of the numerical models were not possible due to a lack of prototype data, it is recommended that a sediment sampling and monitoring program be developed and implemented for Mission Creek. This program would conceivably consist of collecting suspended and bed-material samples from Mission Creek during flood events. Additionally, significant changes in the channel geometry following flood events should be documented in terms of detailed topography. Results from the sampling program could be compared to computed results and used to adjust and verify the numerical models and decrease the uncertainty in the rates and volumes of sediment transport.

Based on the analyses conducted for this project, it is recommended that the use of piers or multiple box openings for future bridges be avoided if positive debris control is not provided. The presence of bridge piers in a high-velocity channel tends to obstruct flow and may encourage deposition of sediment. The use of clear-span bridges is therefore recommended.

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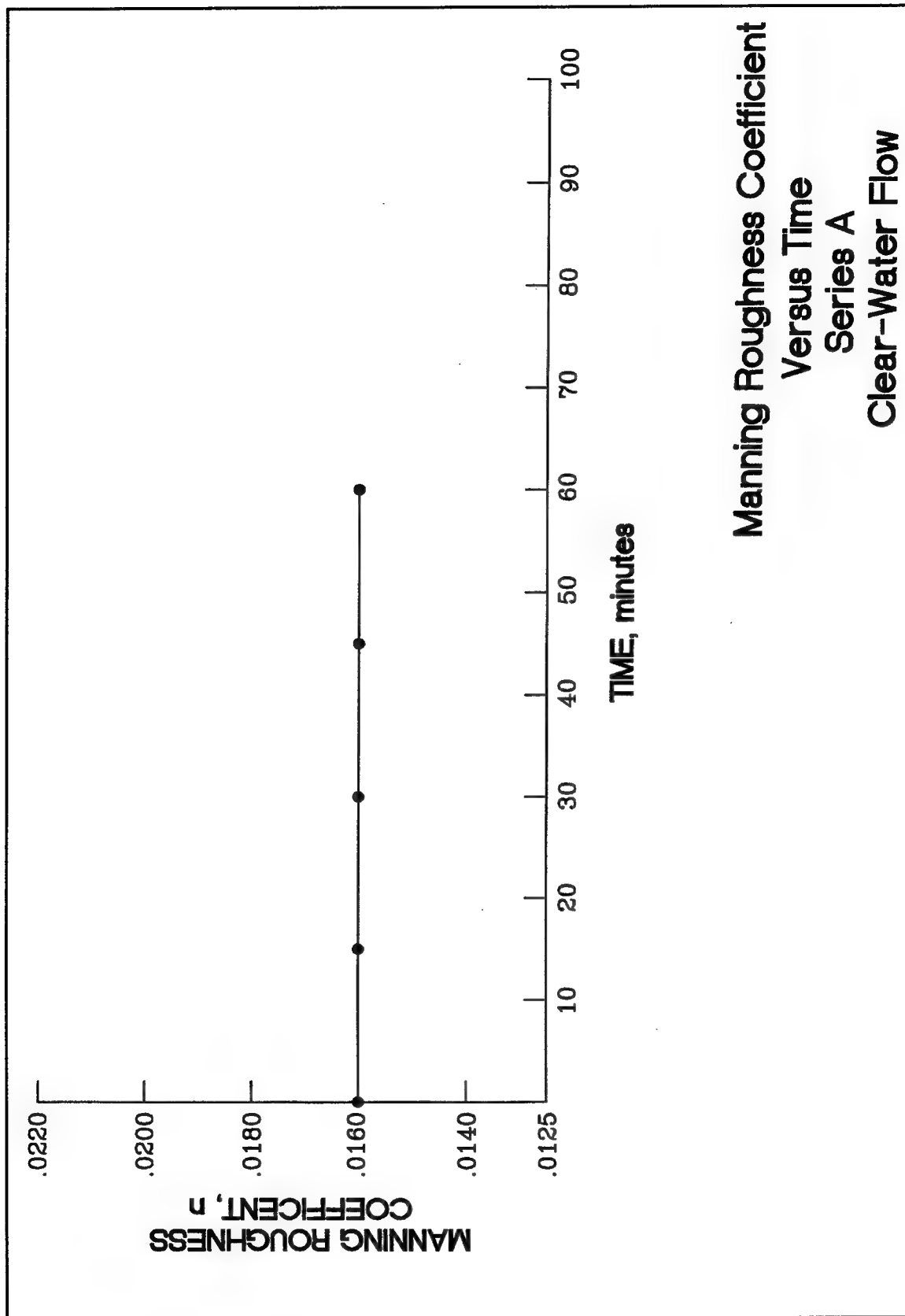
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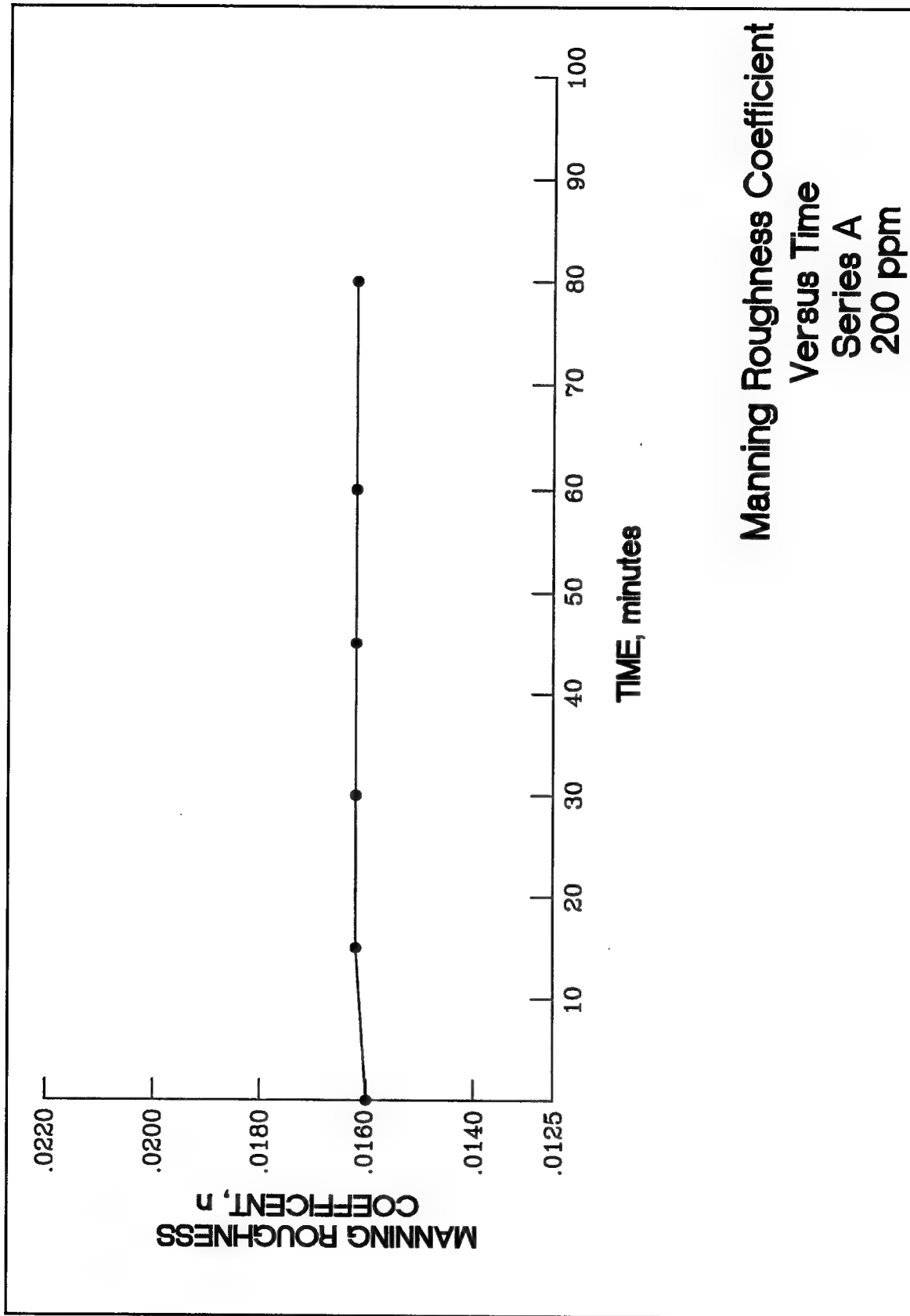
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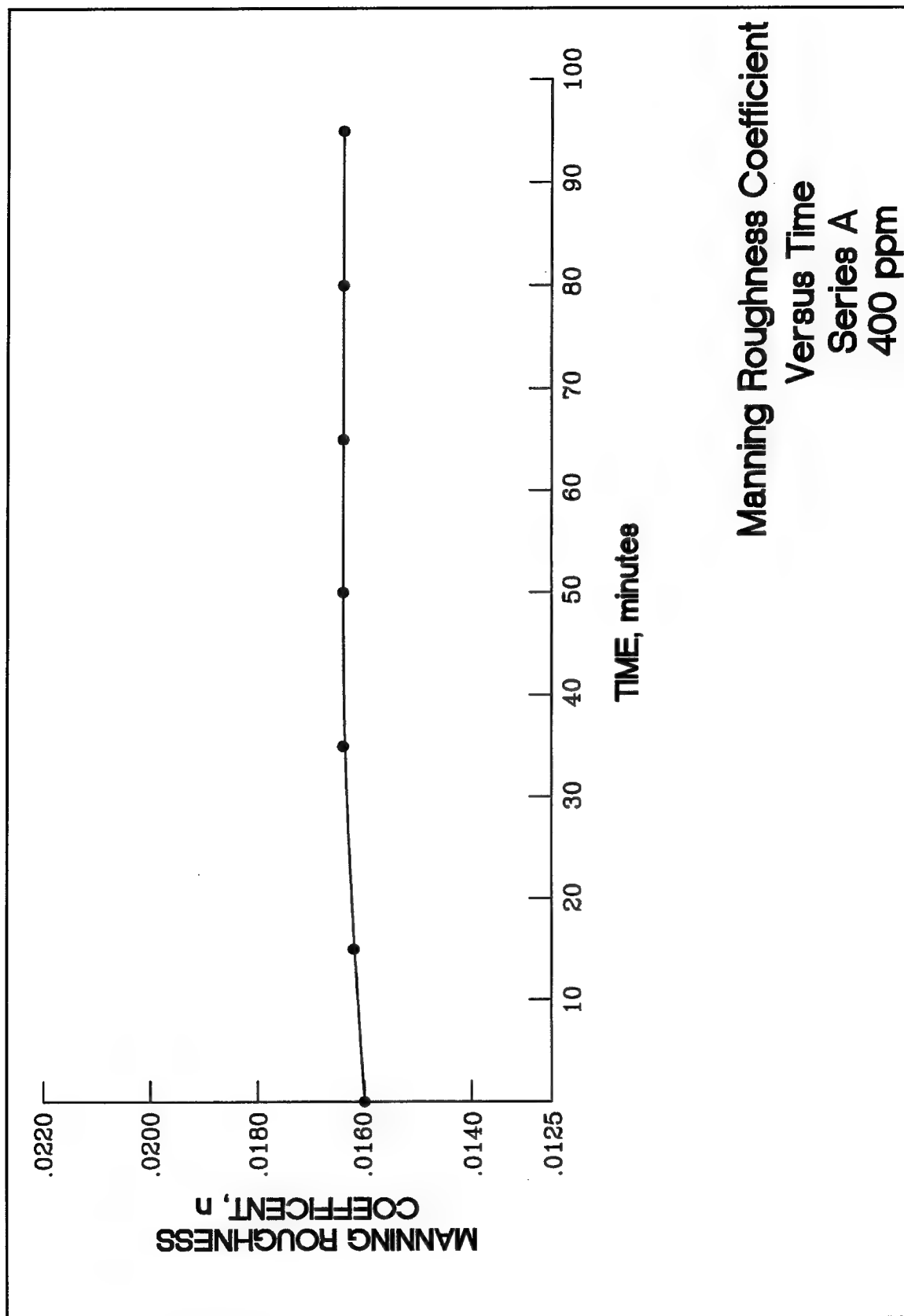
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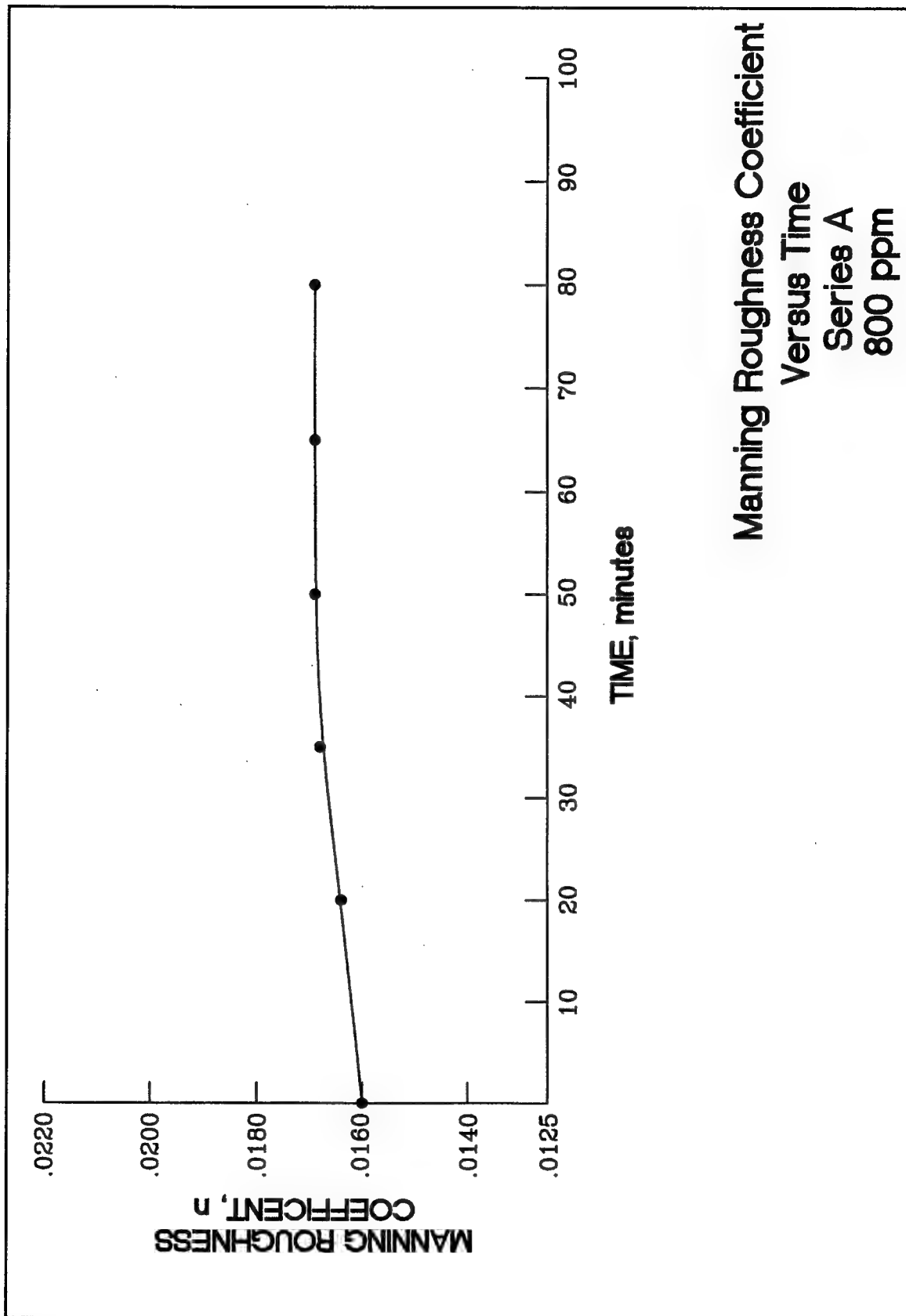
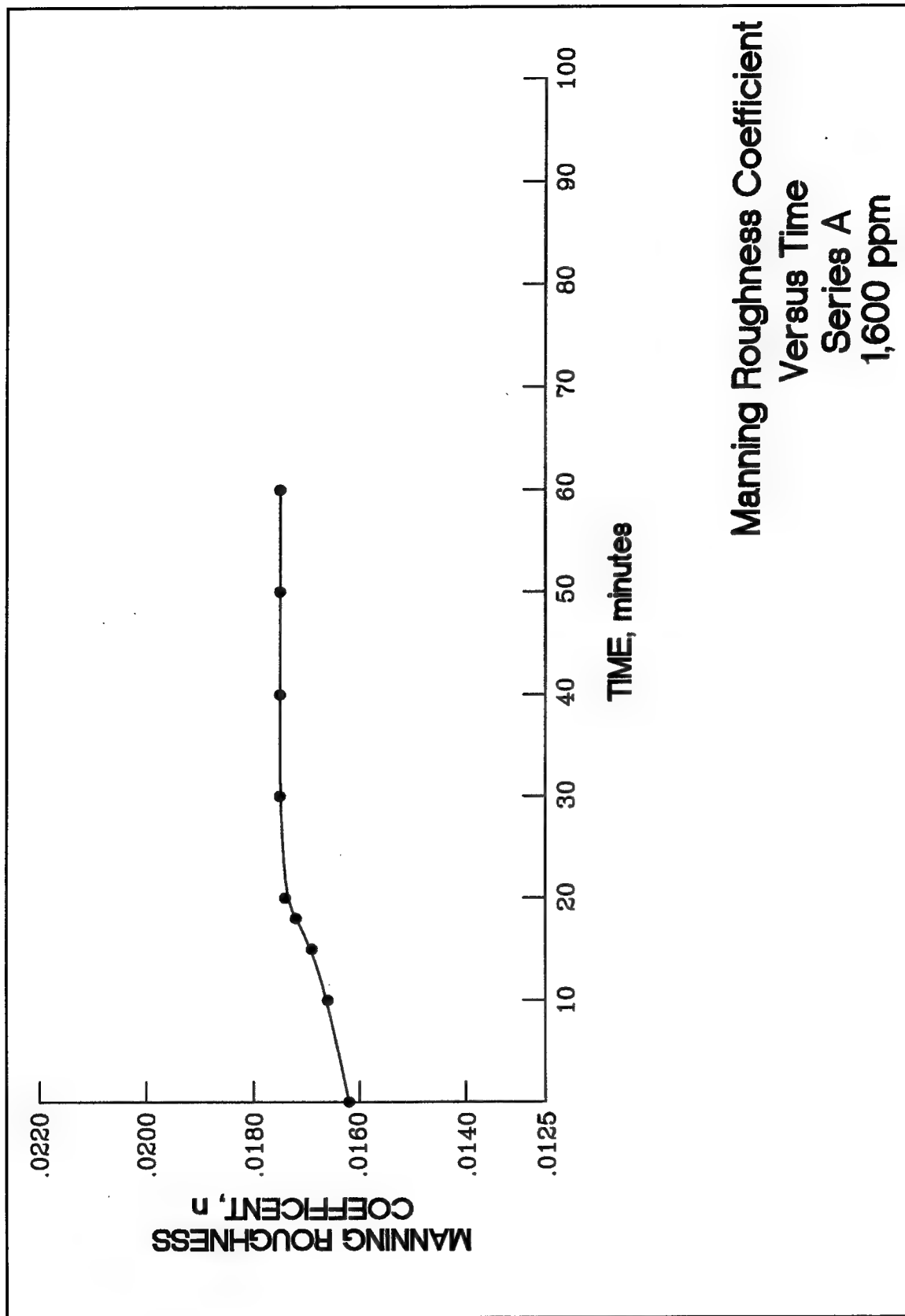
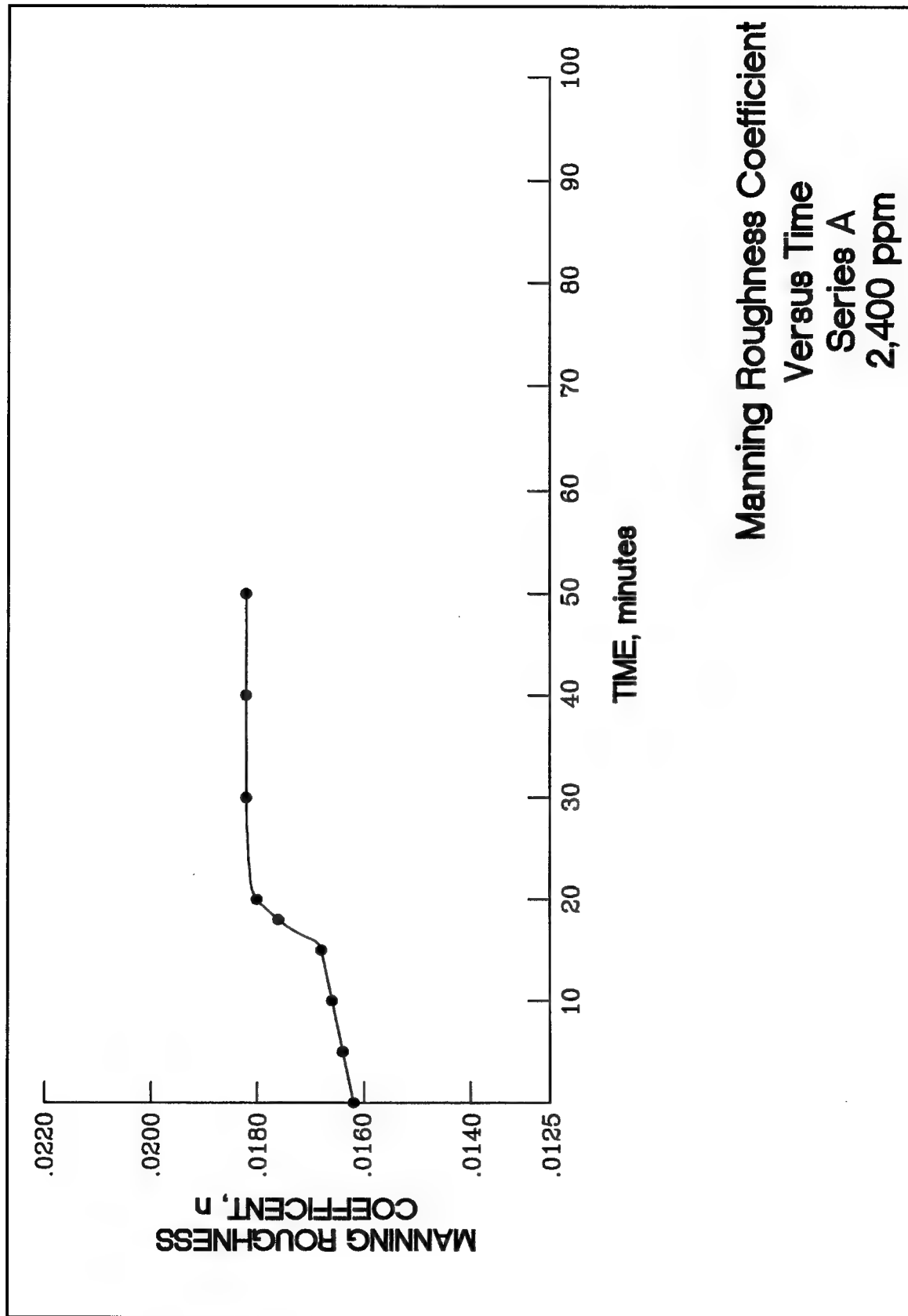
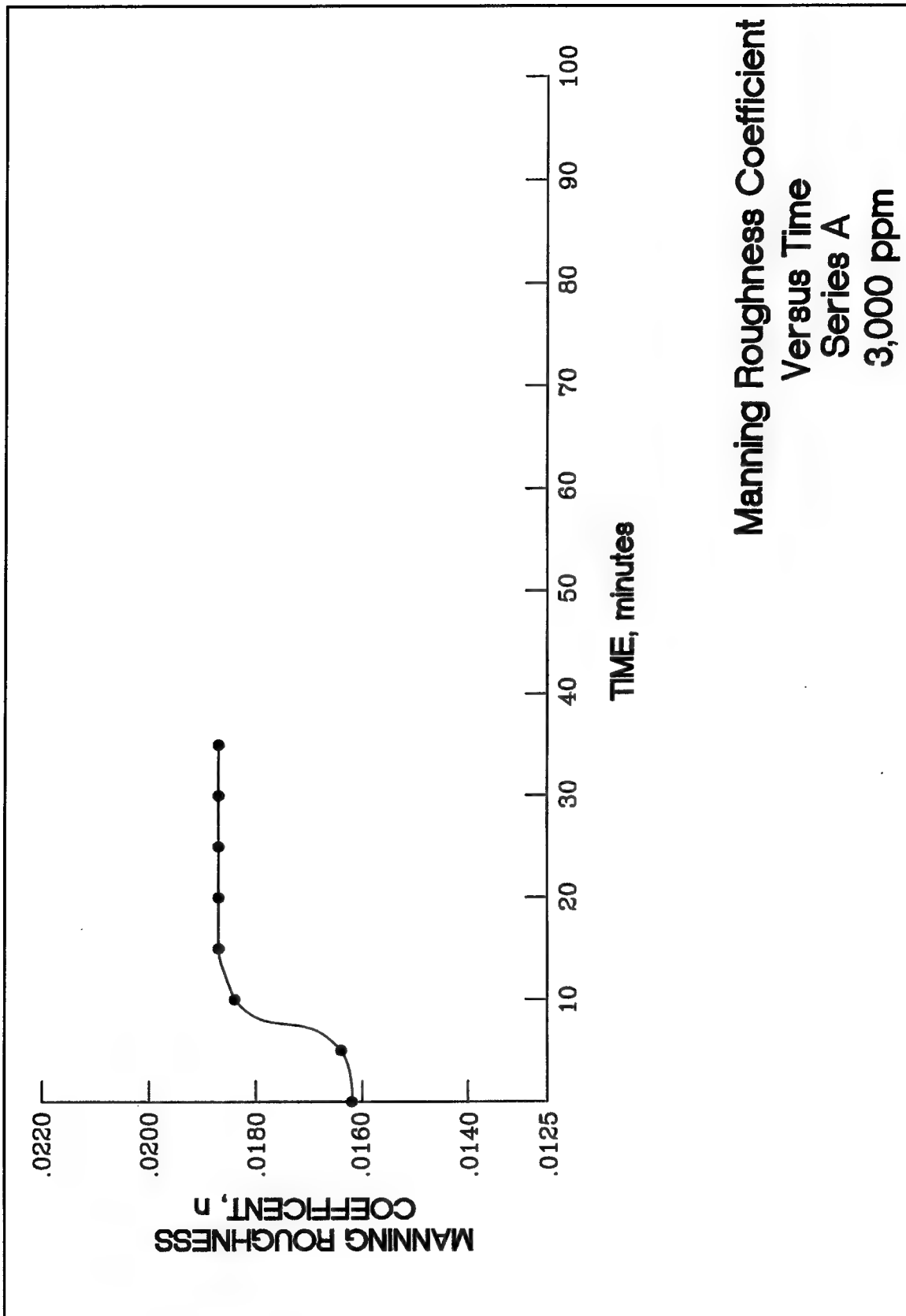


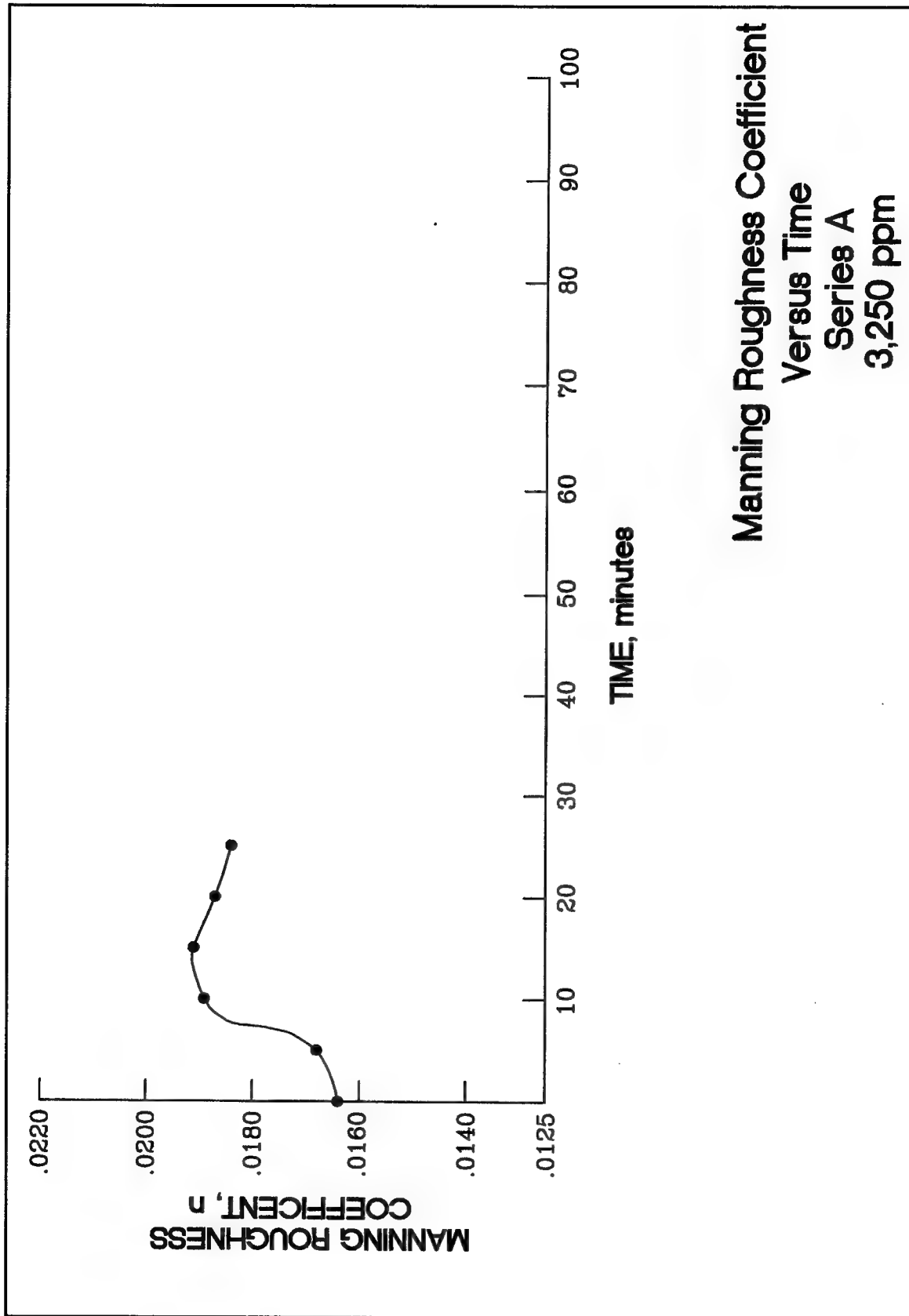
Plate 4

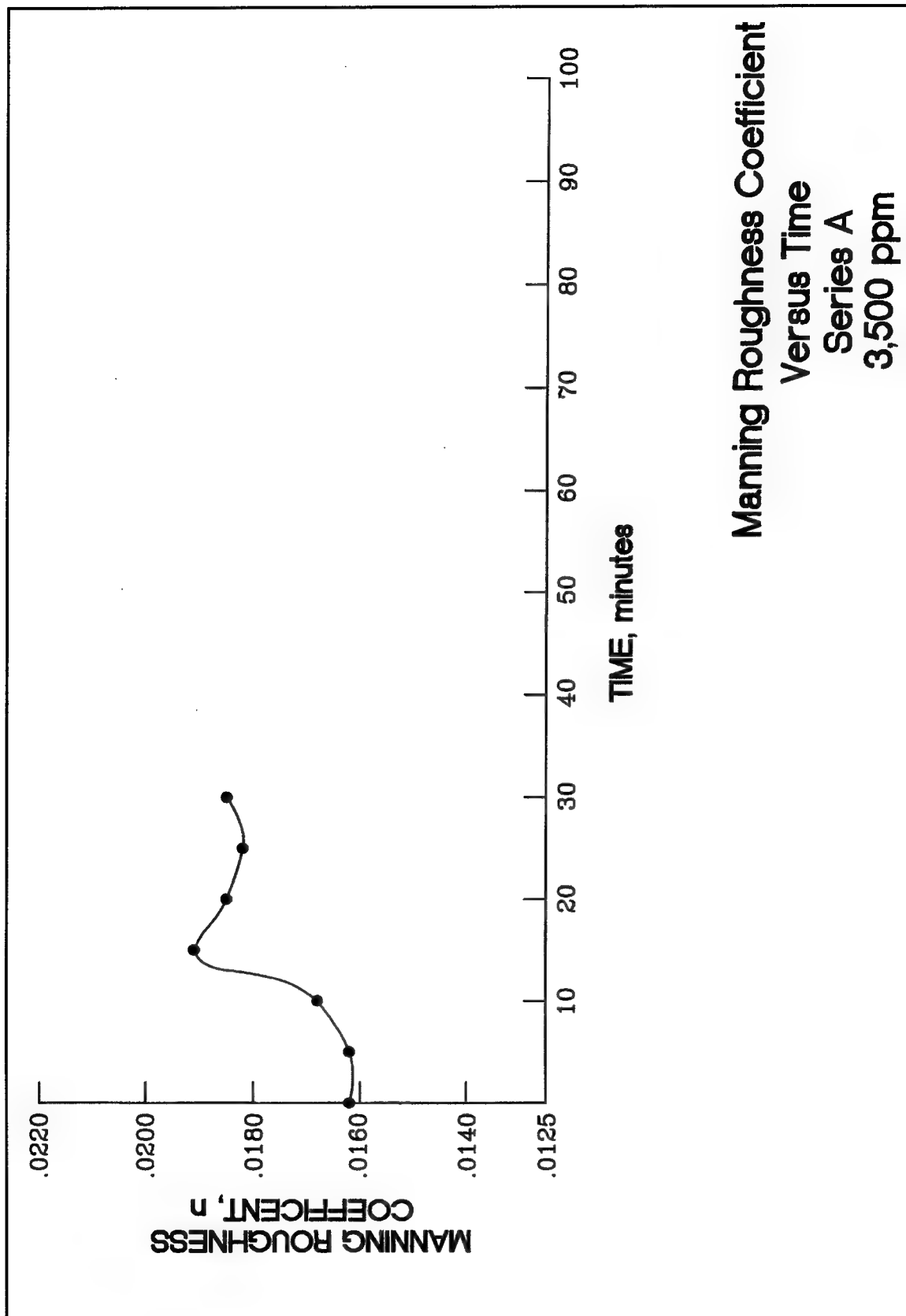


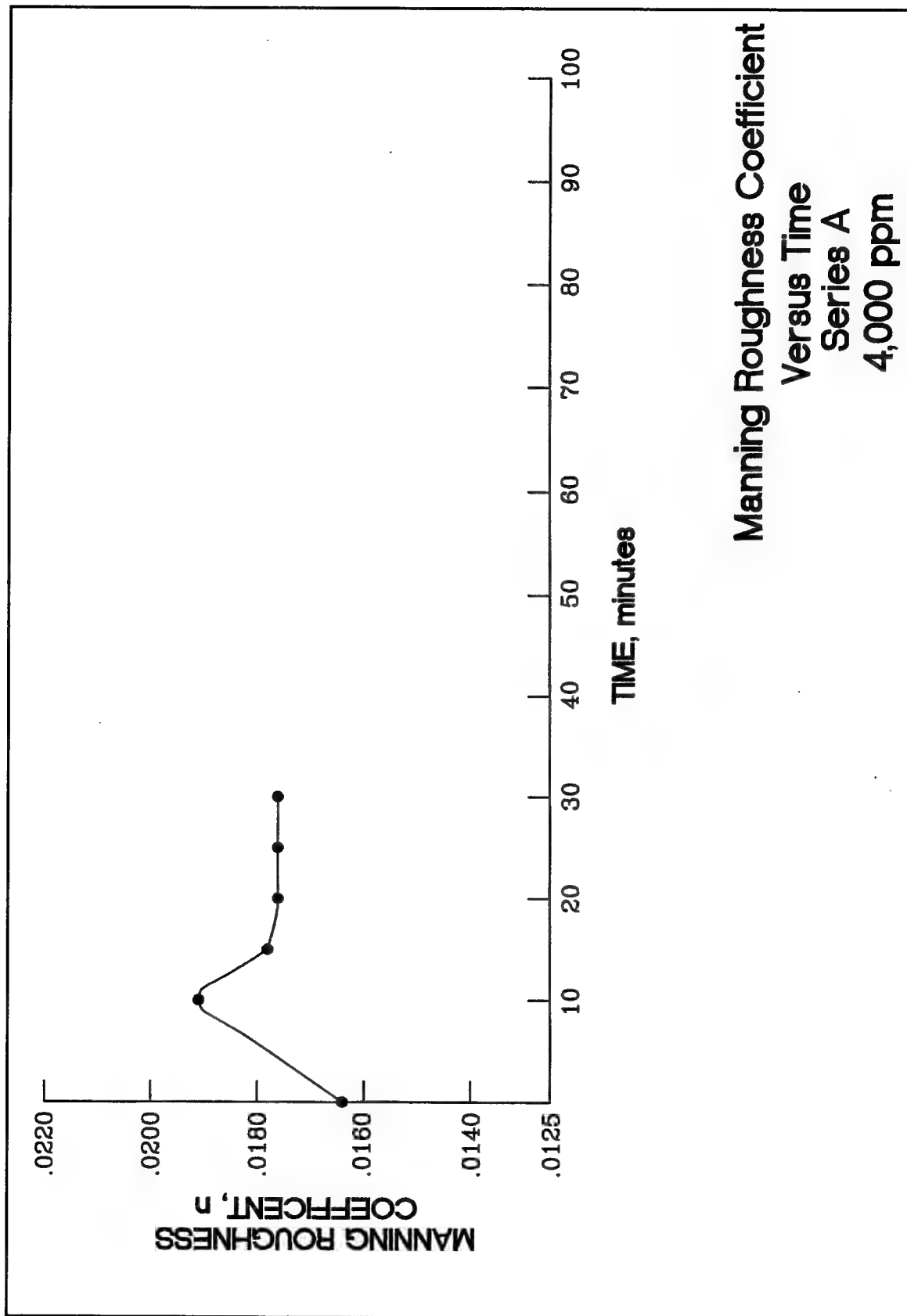


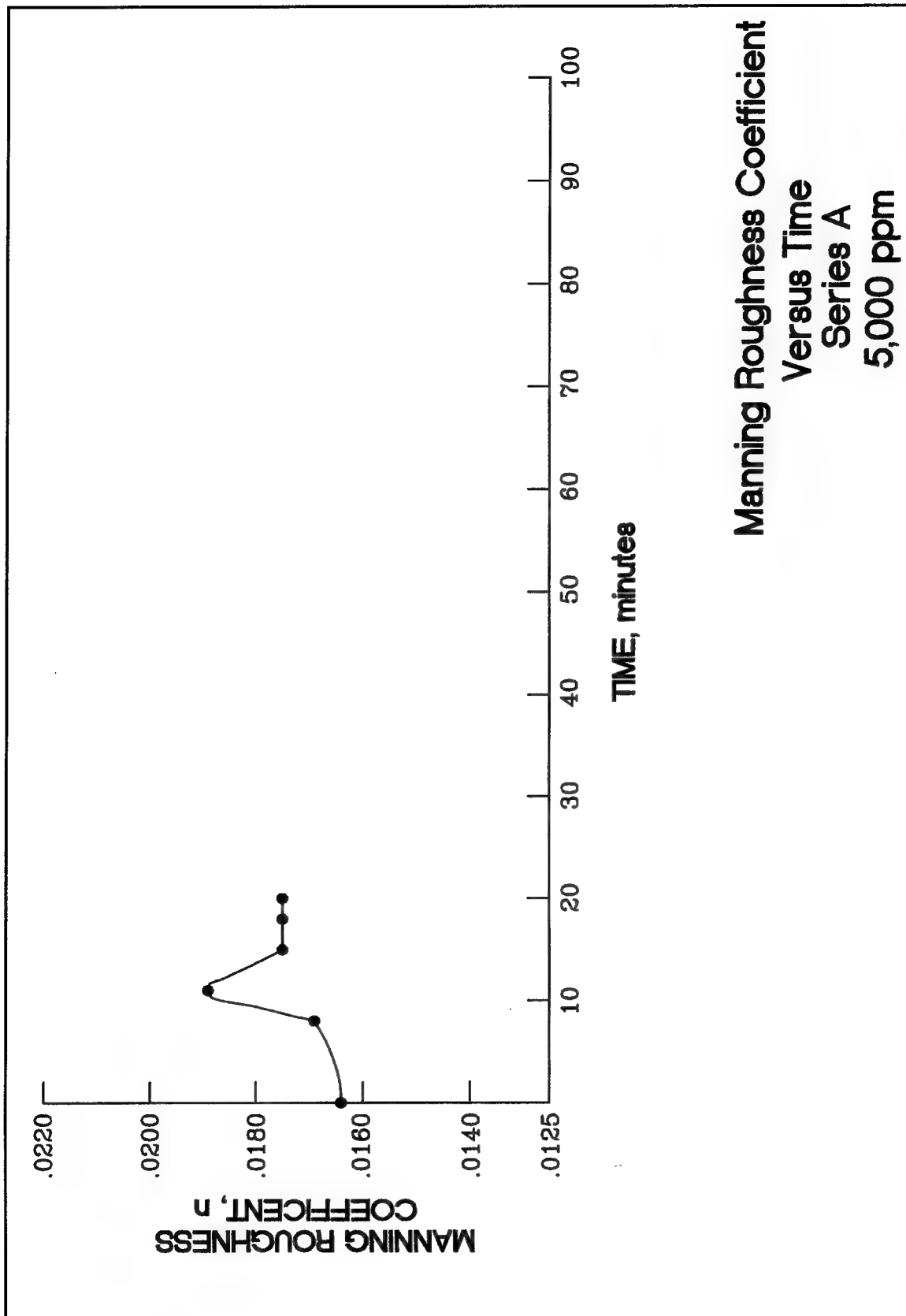


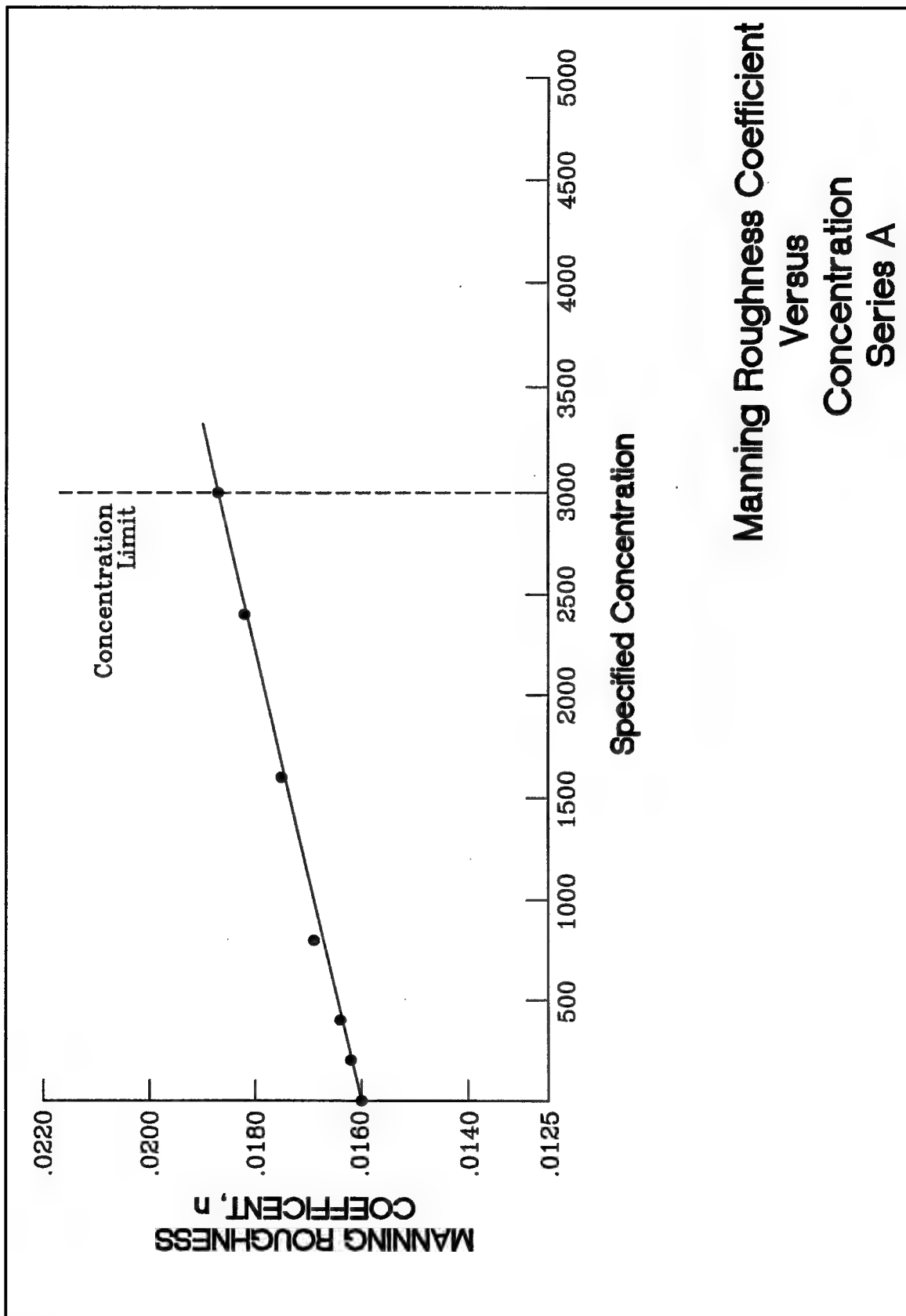


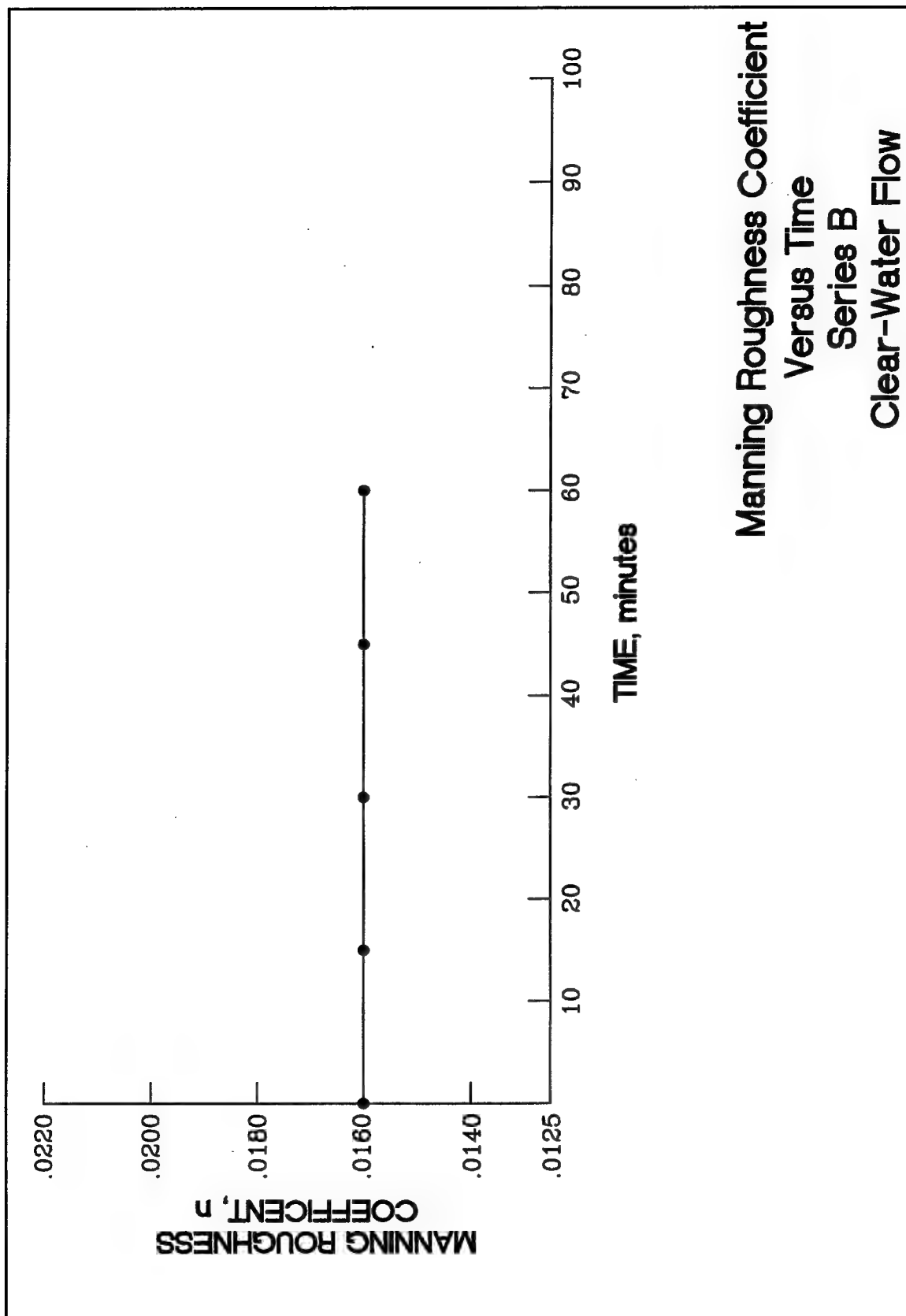


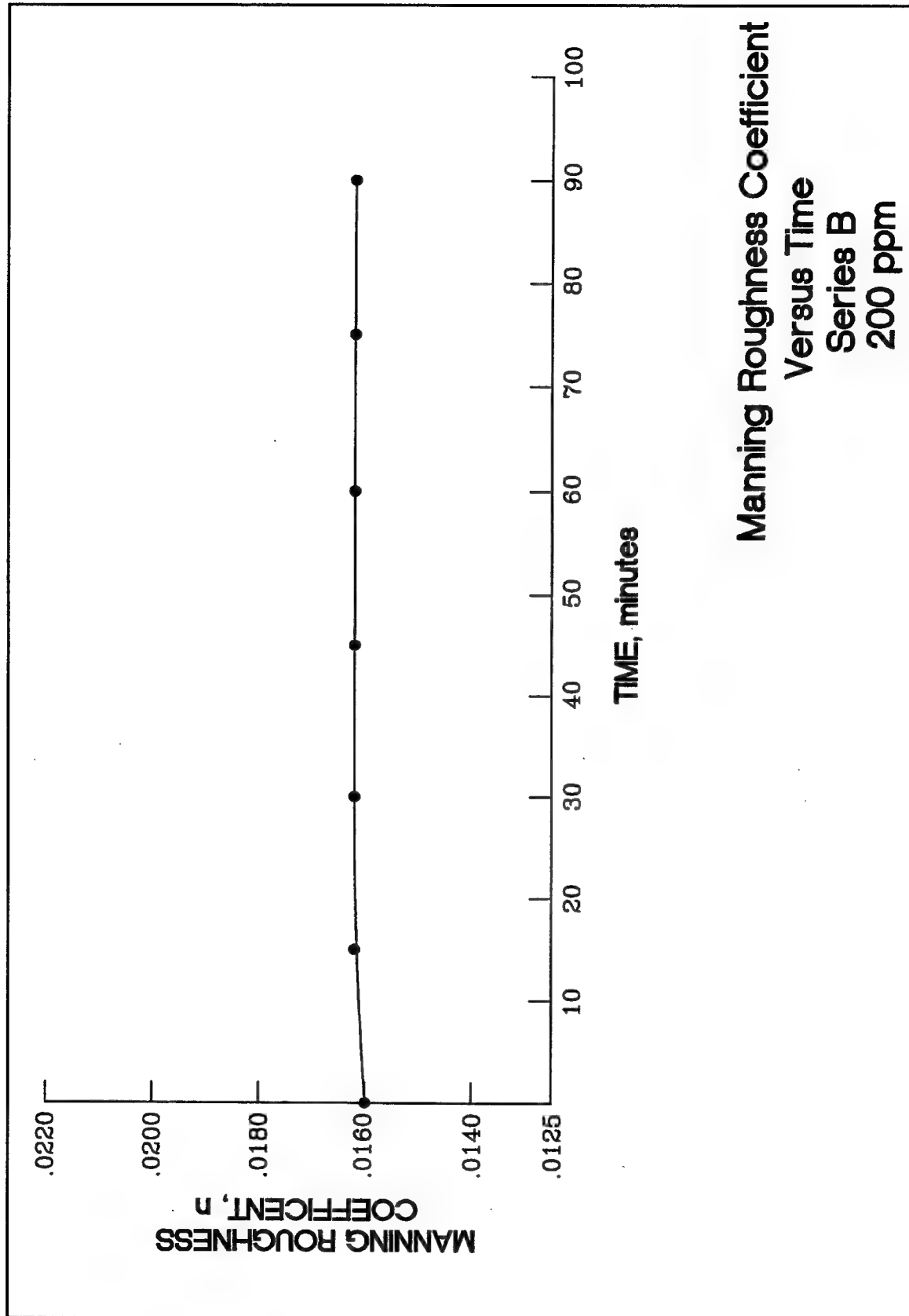


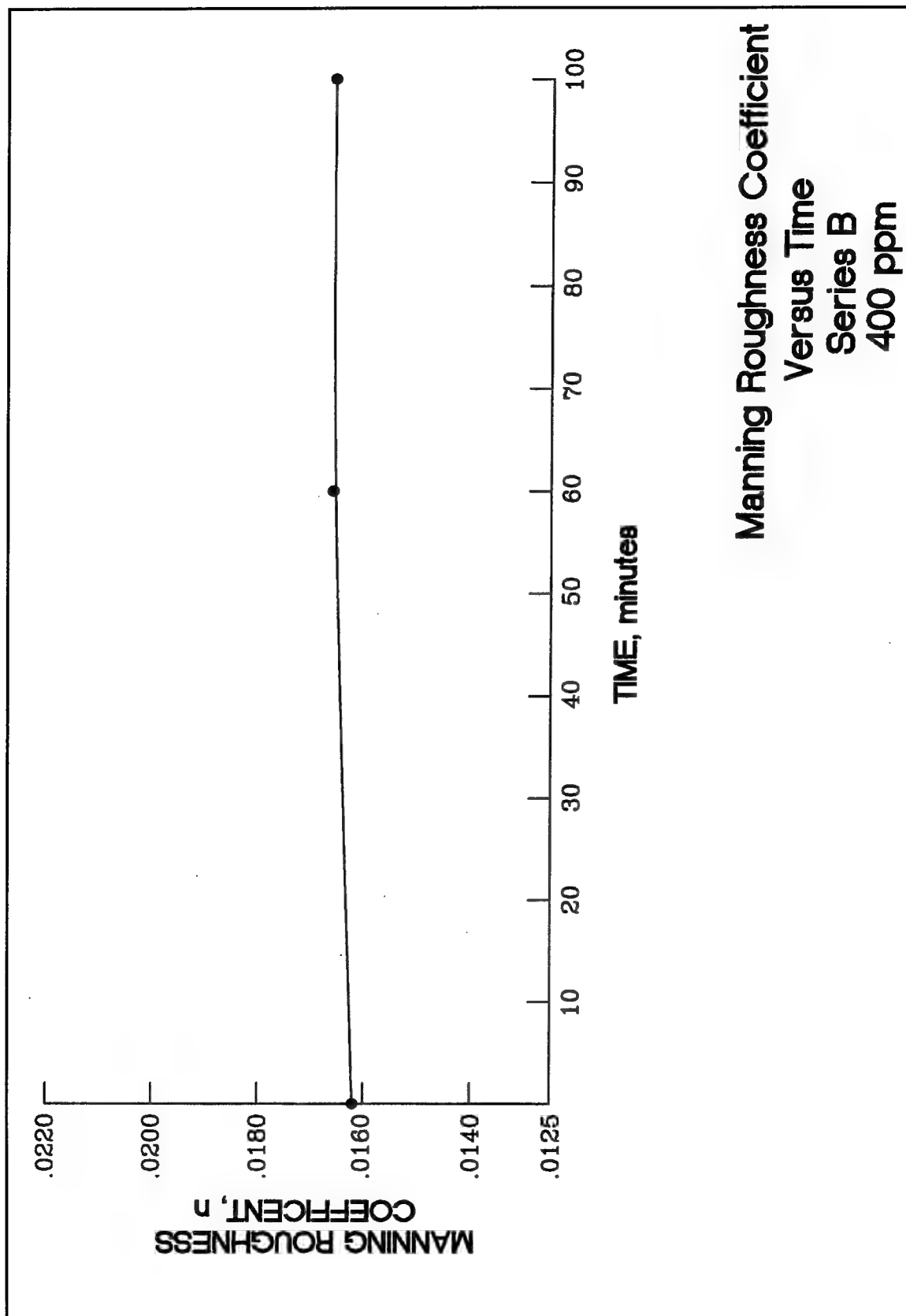




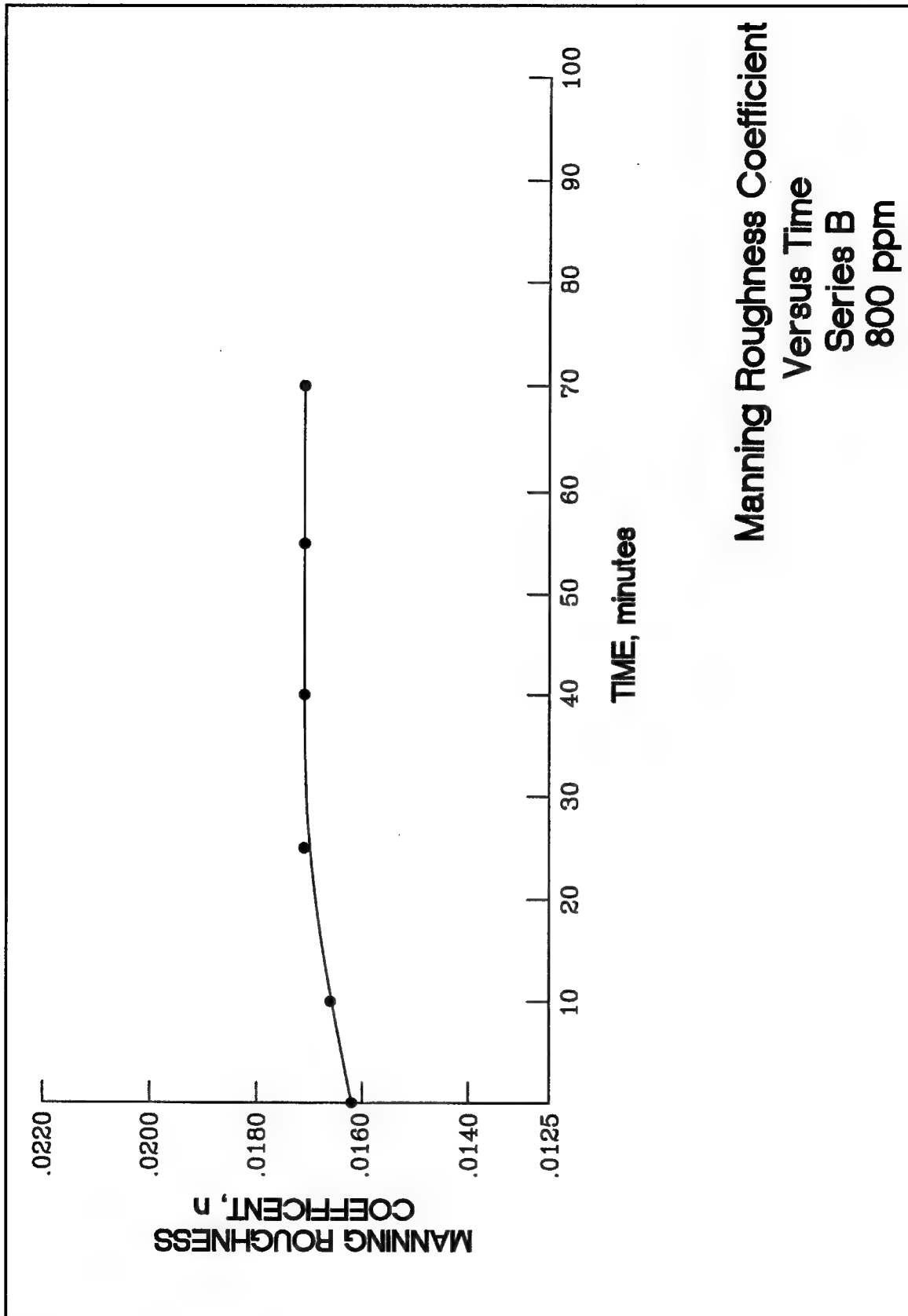


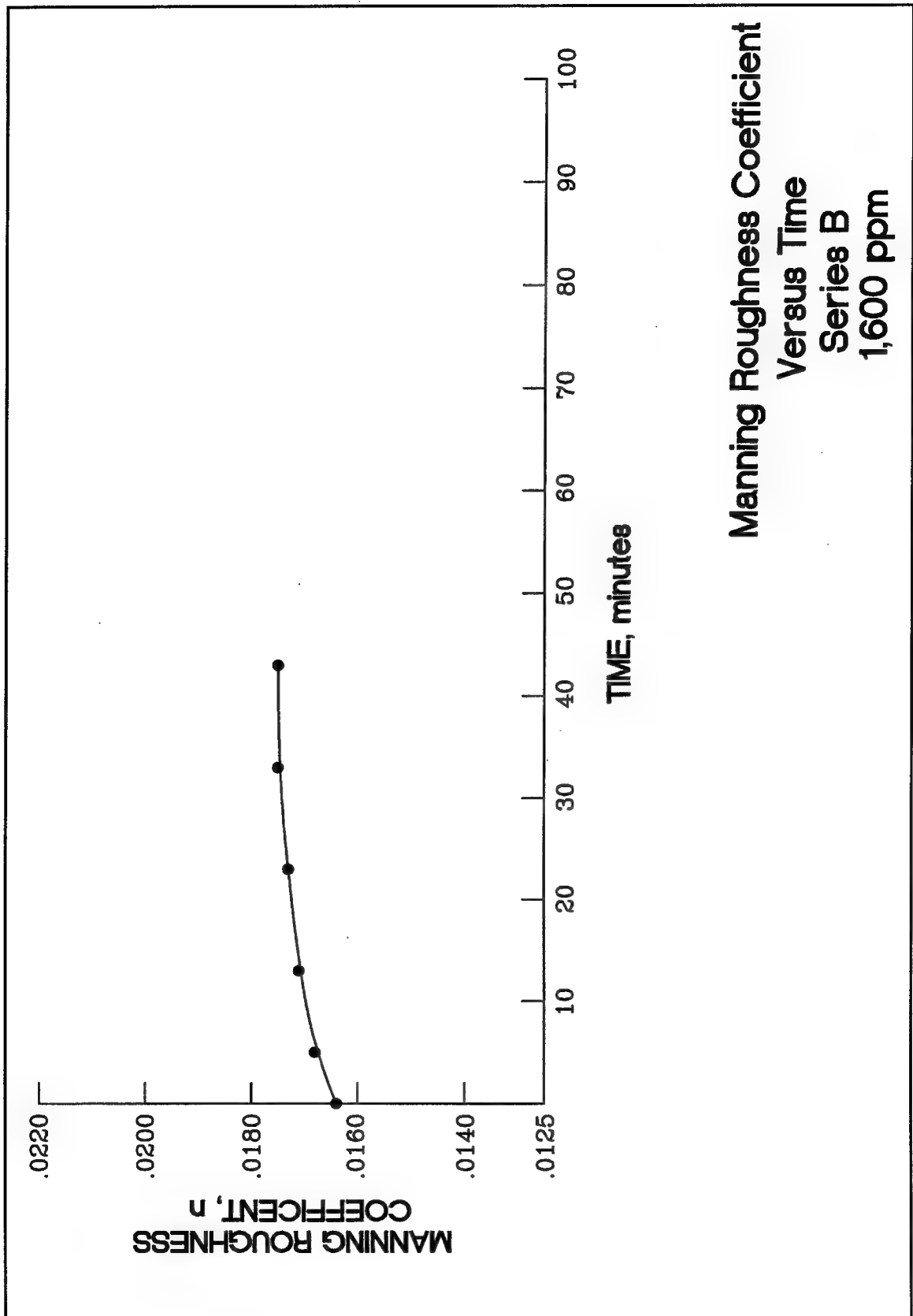


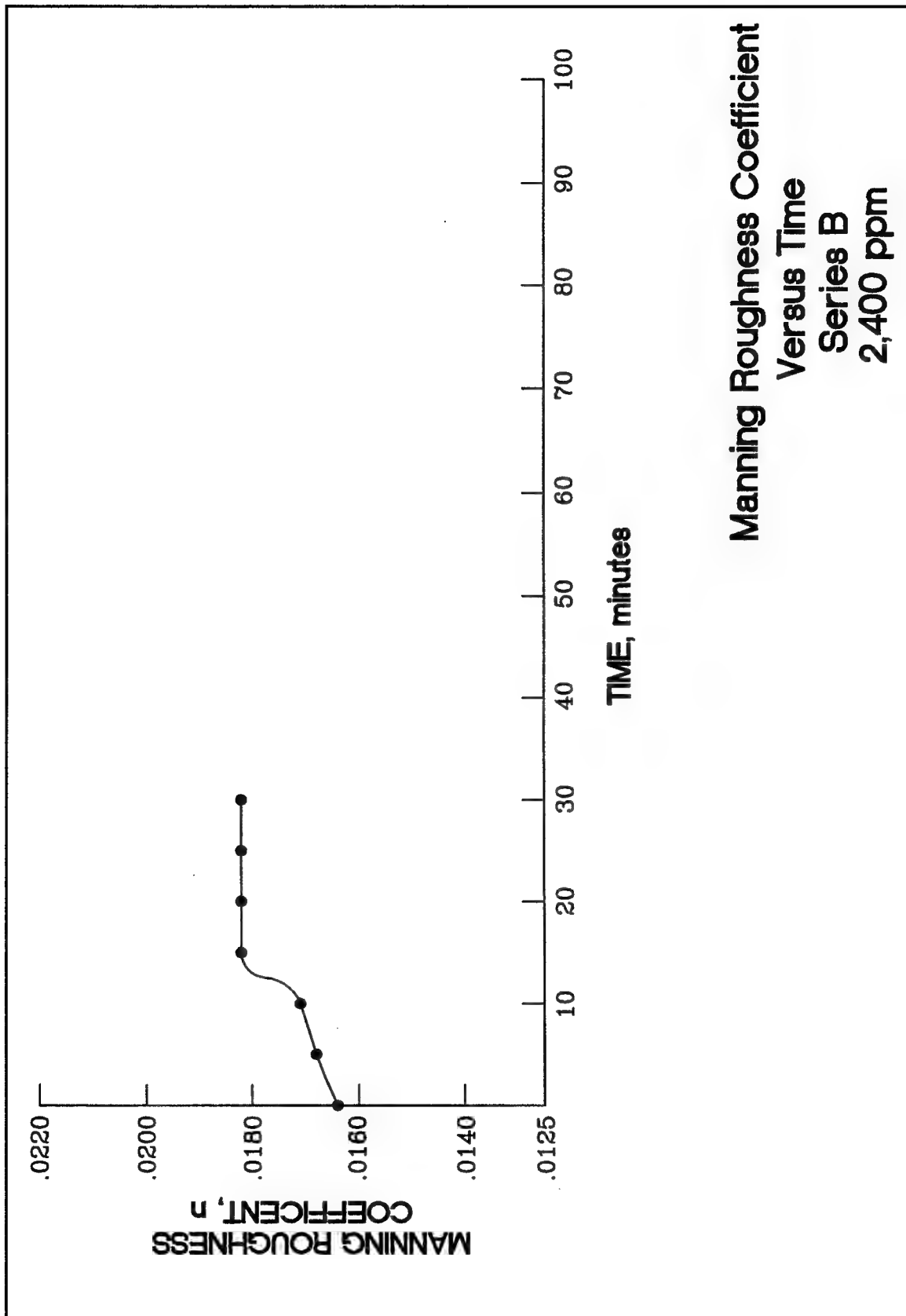


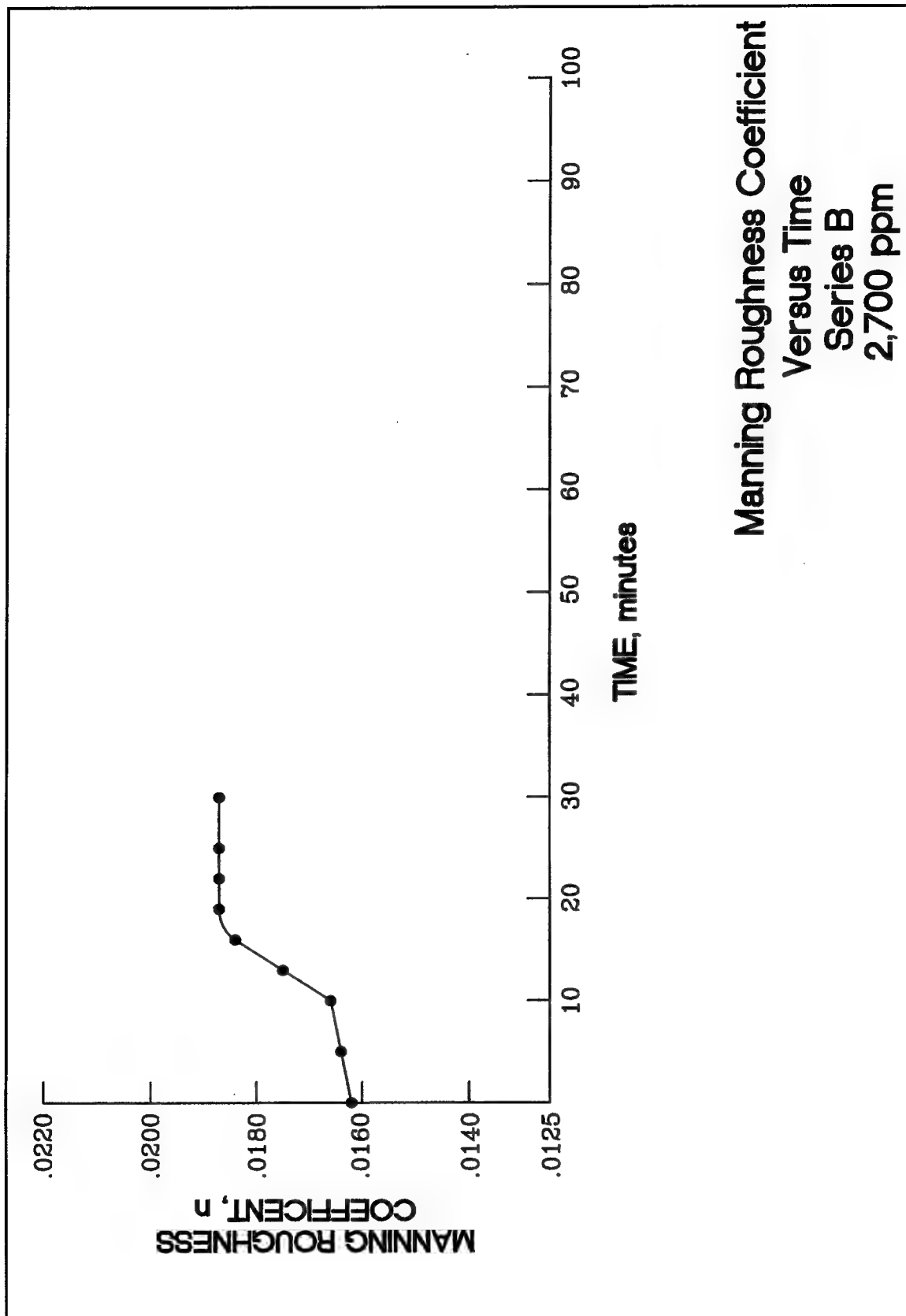


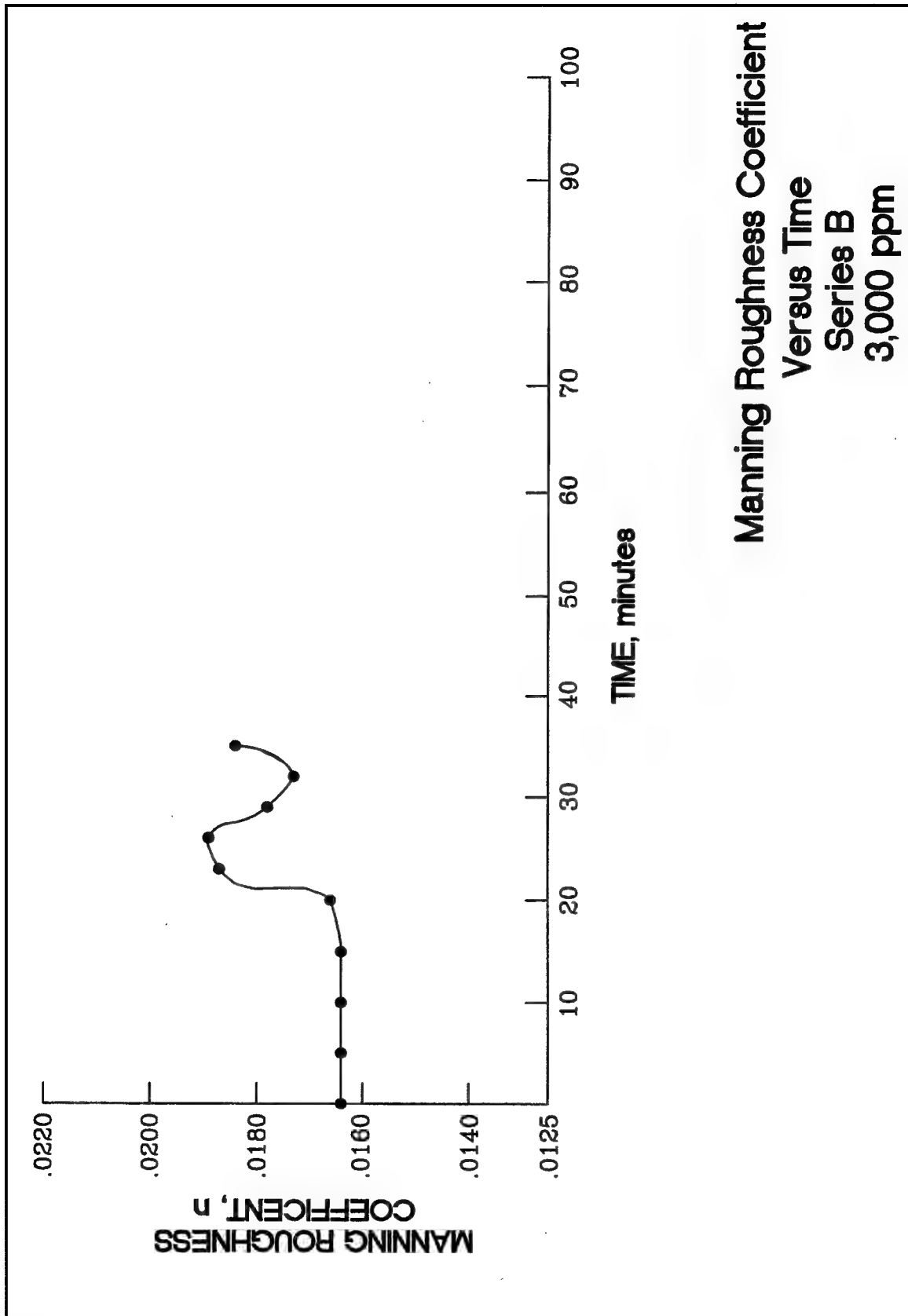


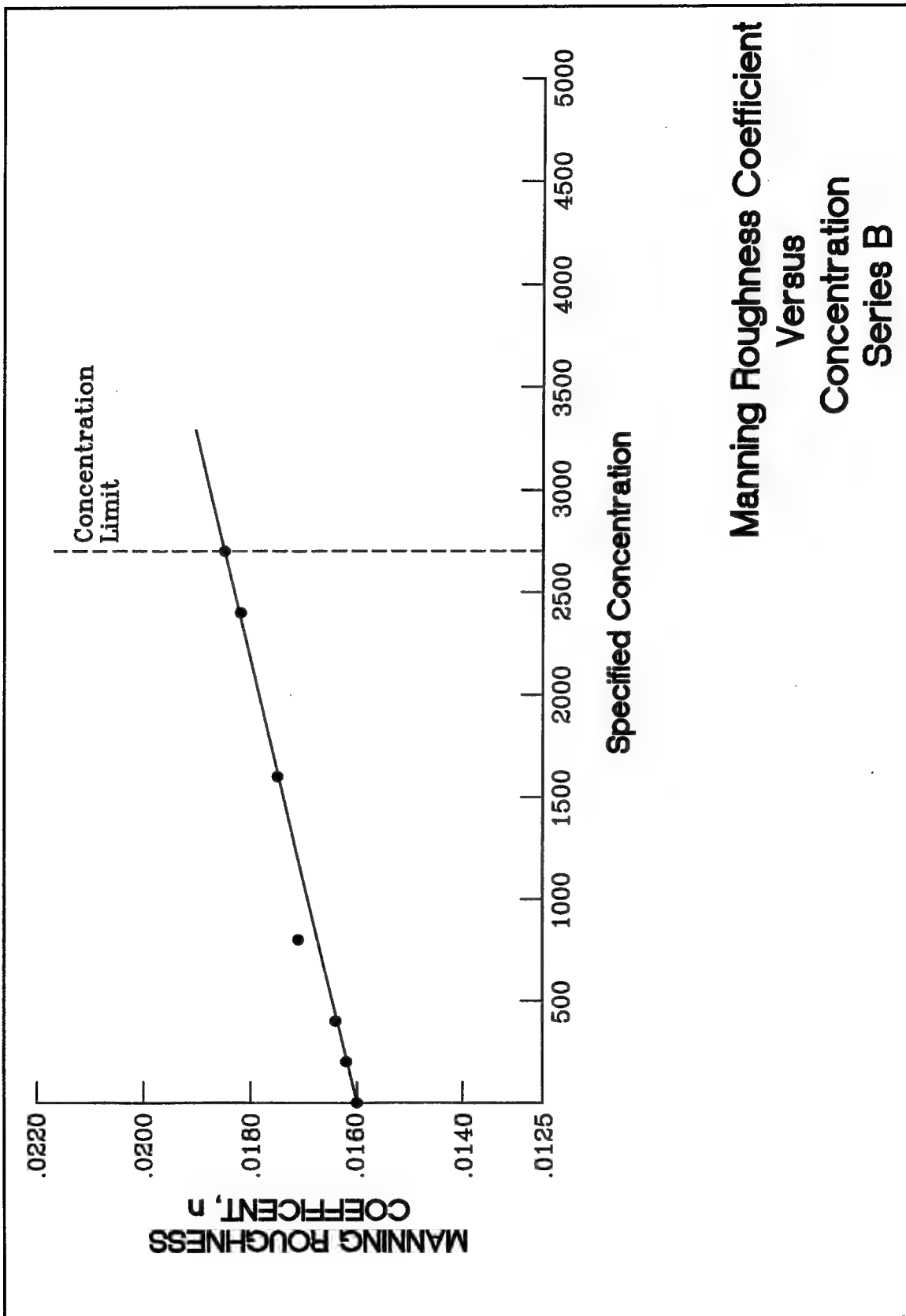


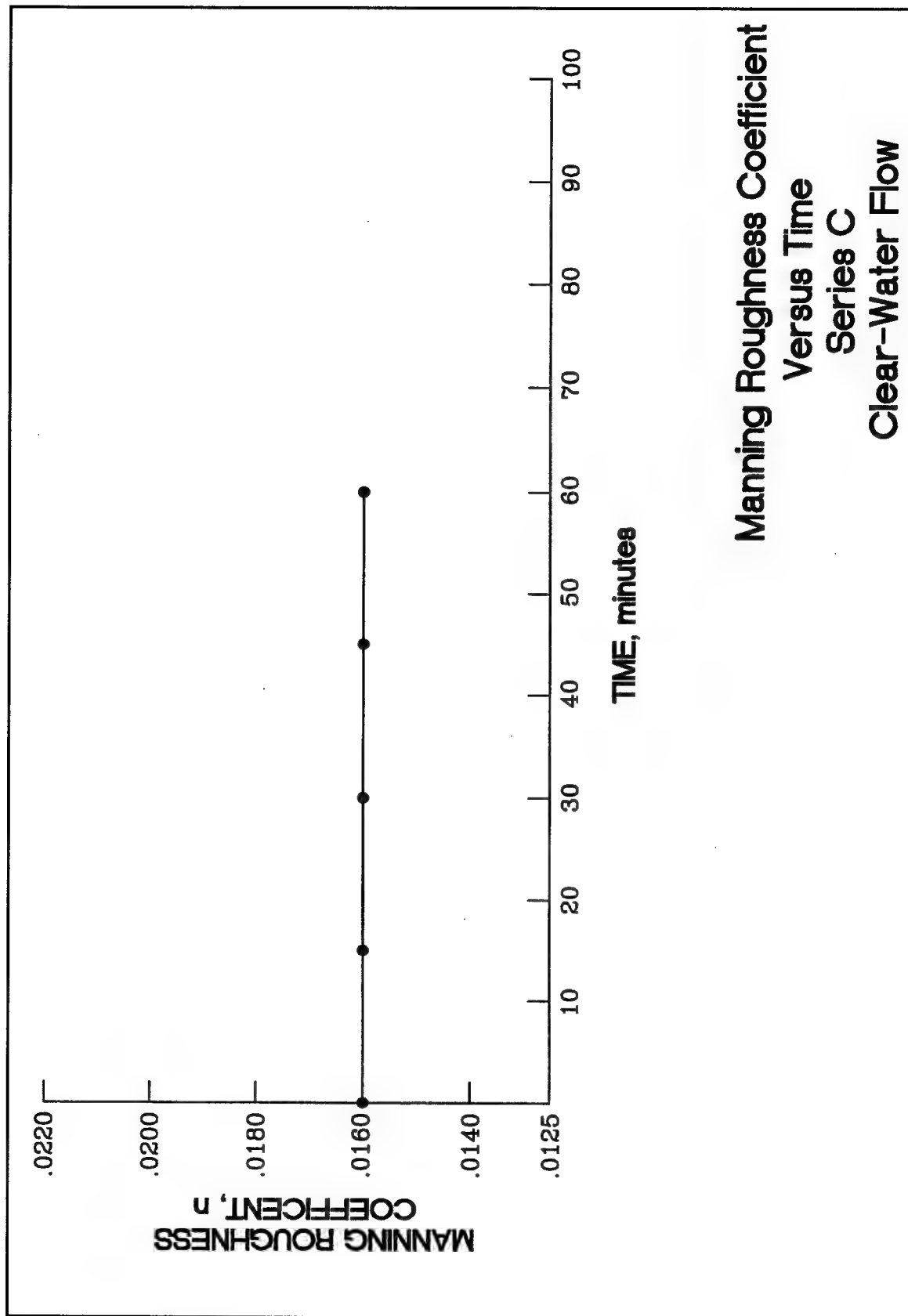


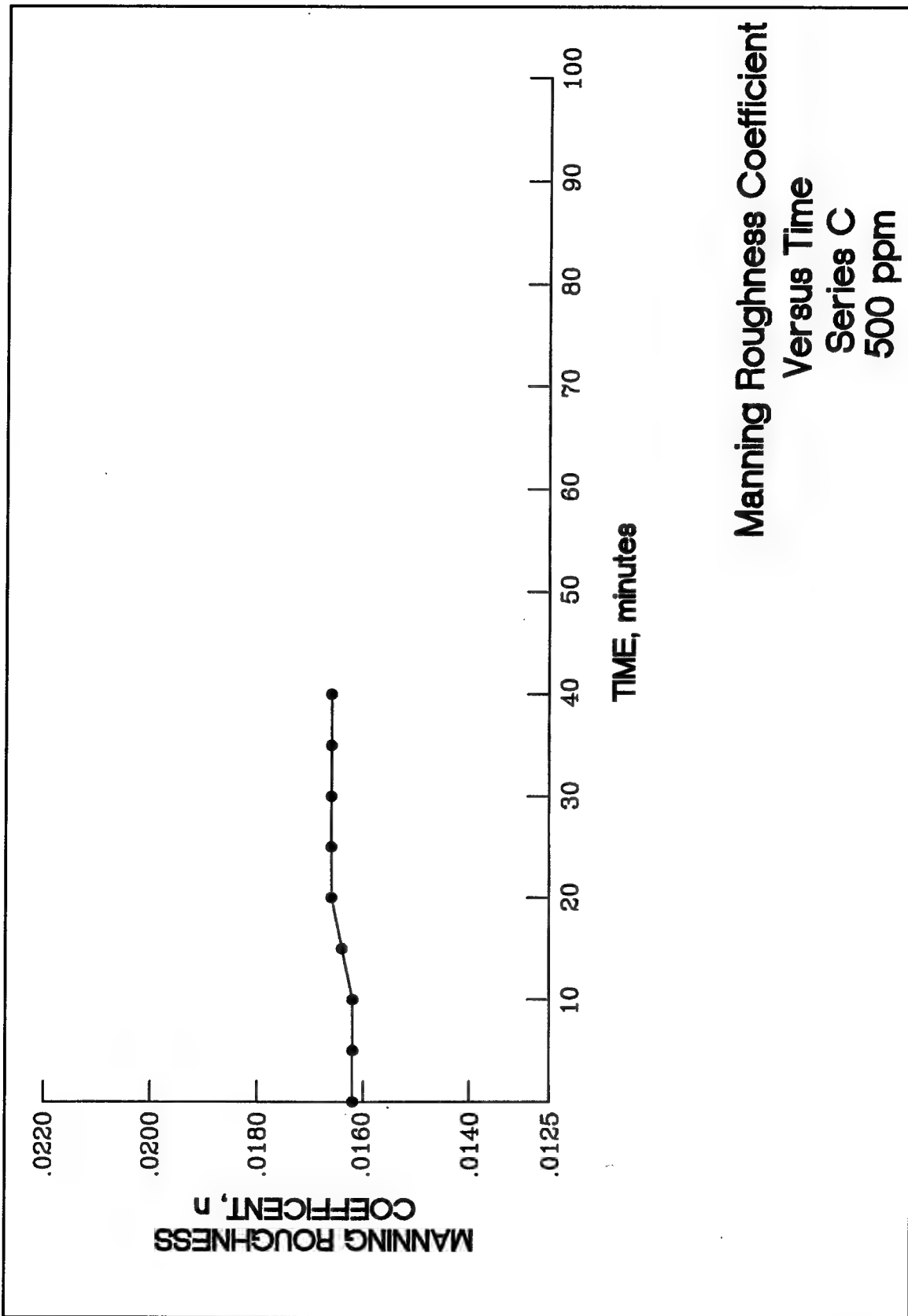




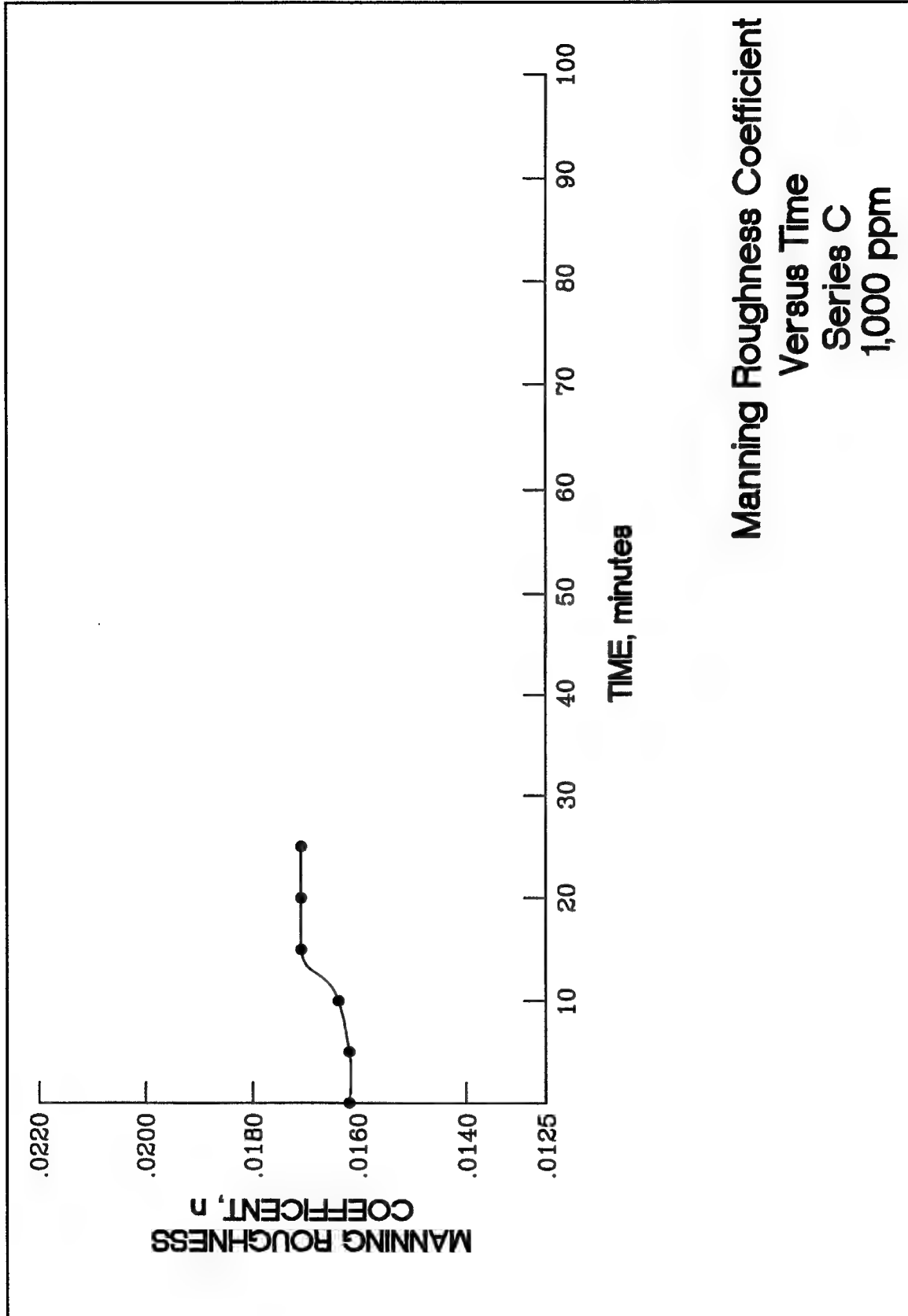


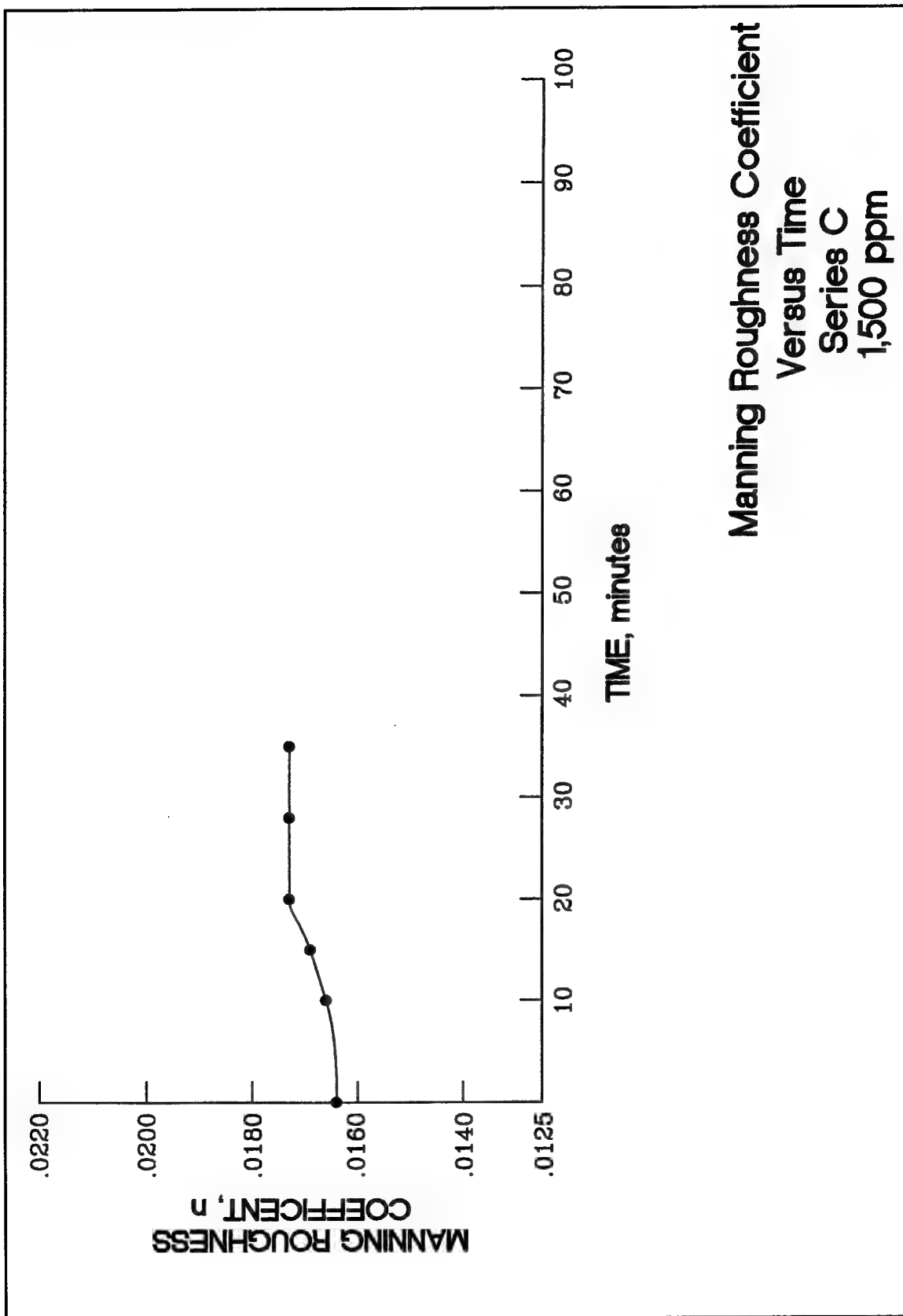


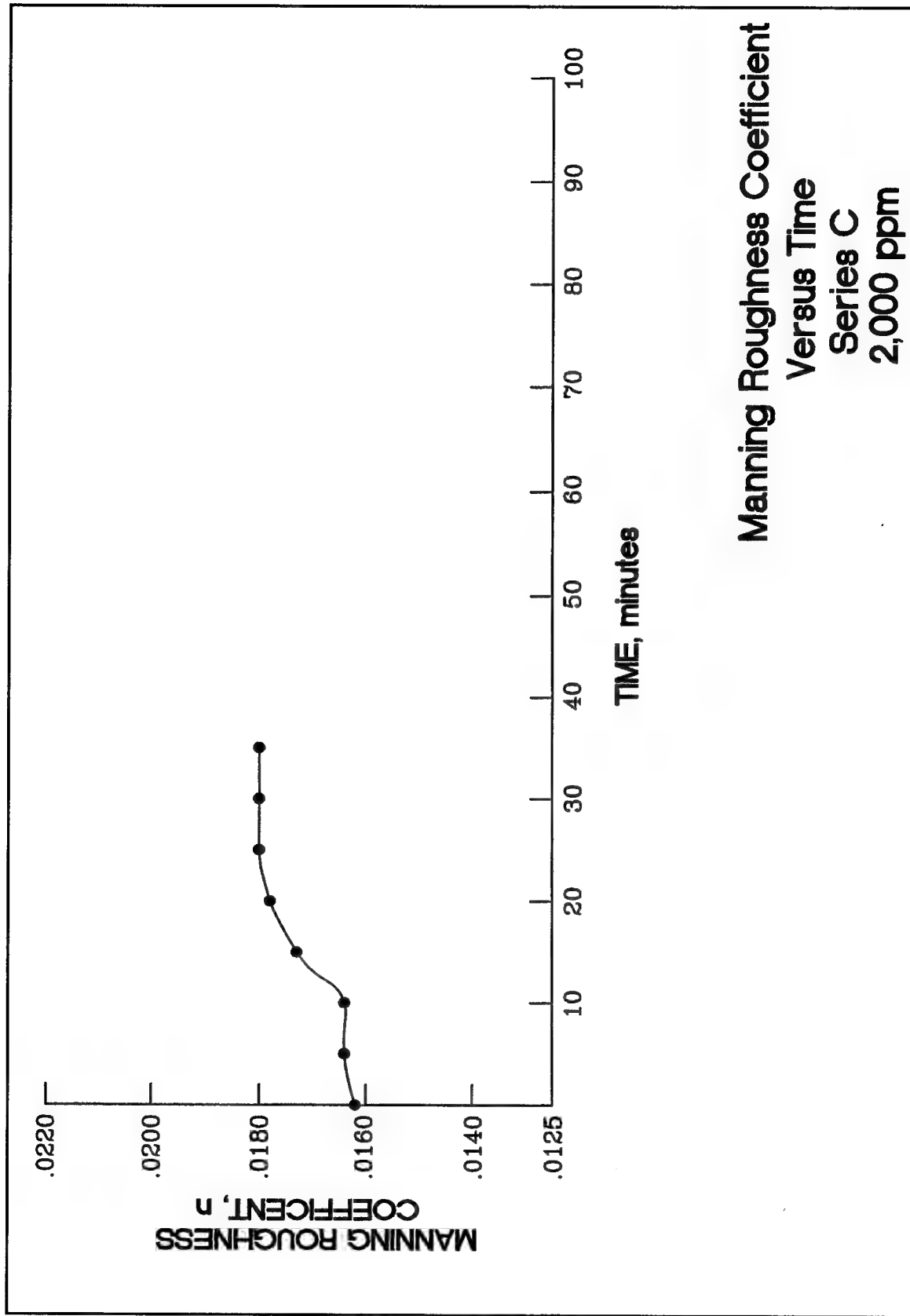


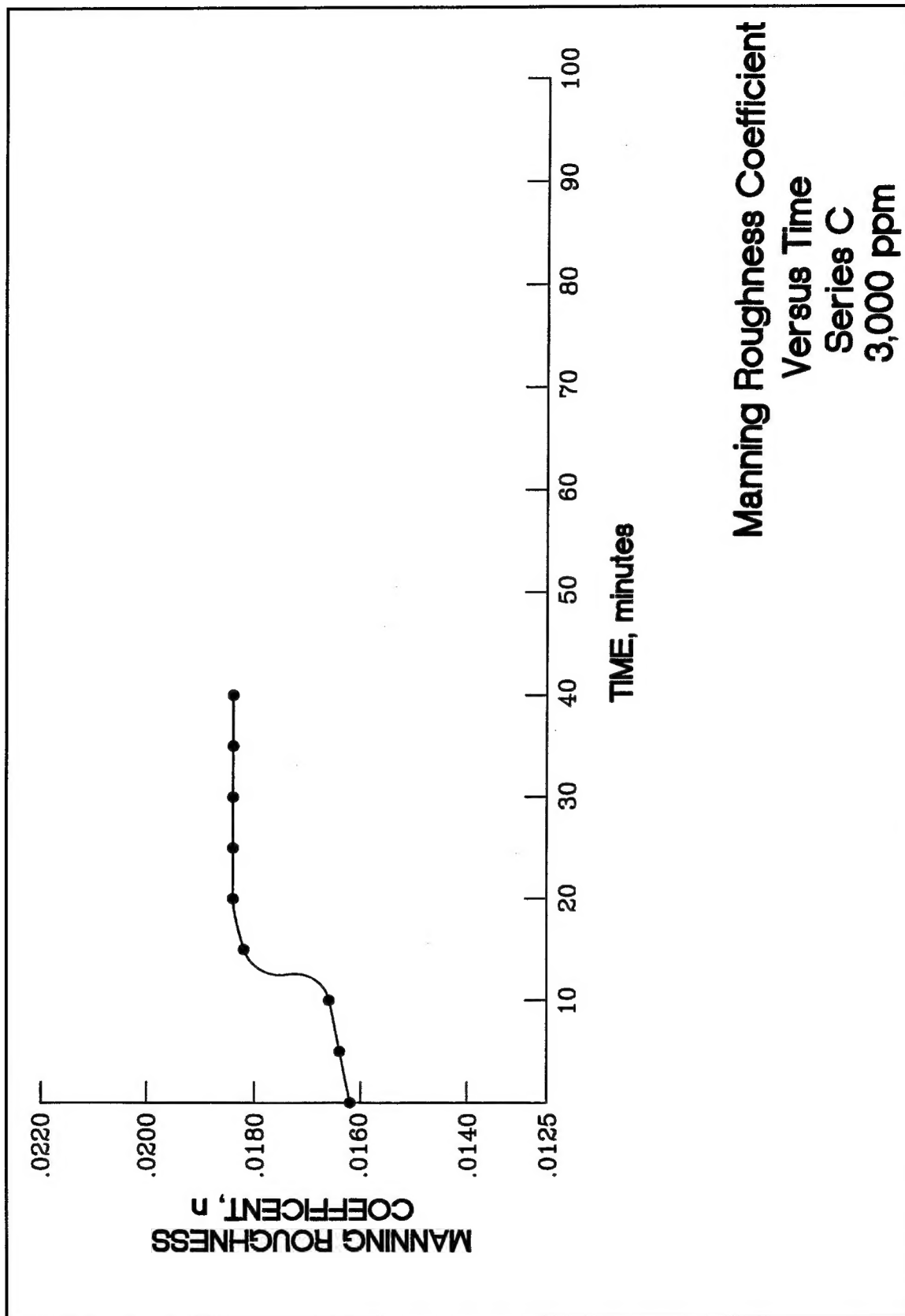


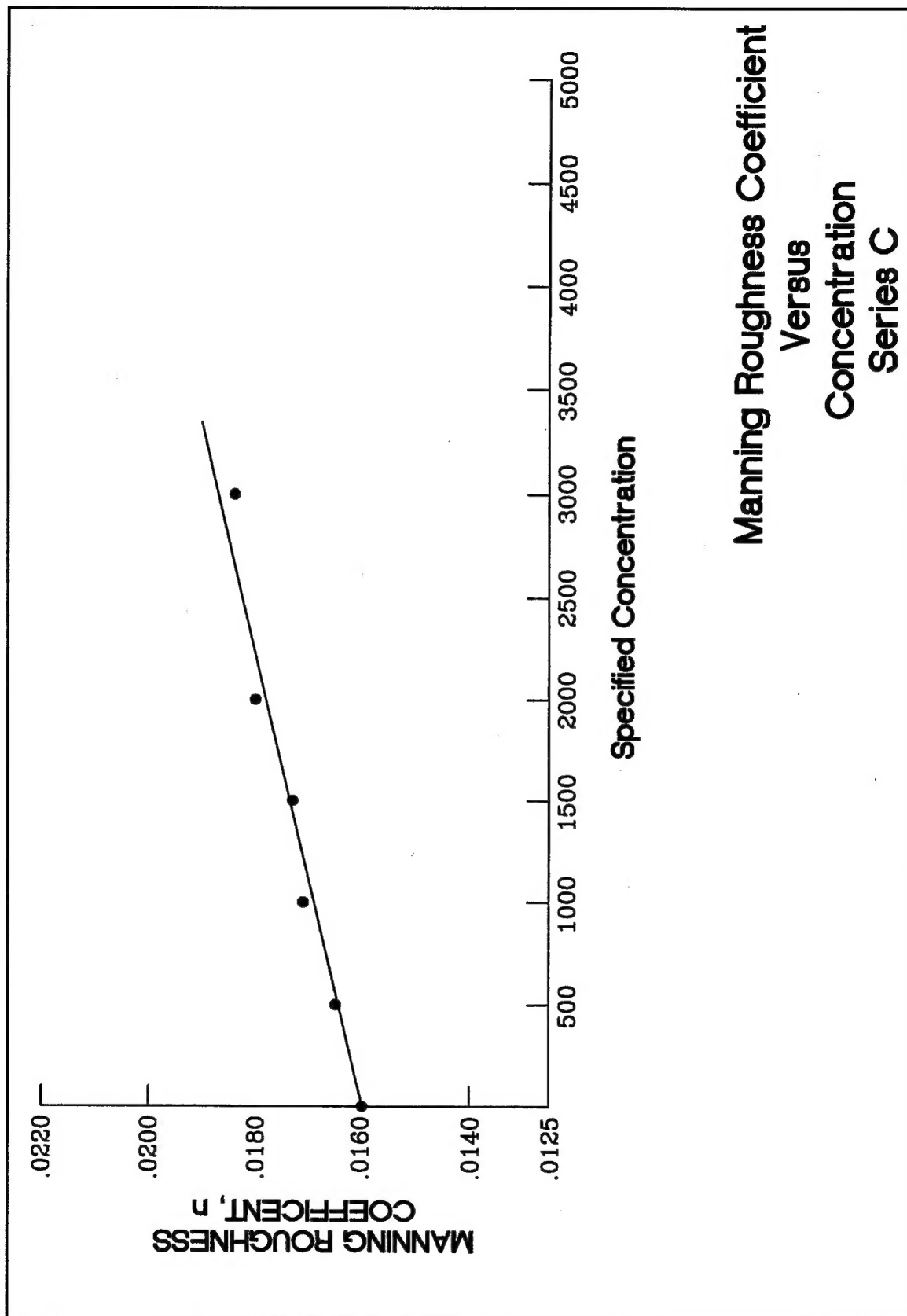












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